Gibbons

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ISSUED QUARTERLY

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OF THE
New England Water Works Association
1953 - 1954

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THE Association was organized in Boston, Mass., on June 21, 1882, with the object of providing its members with means of social intercourse and for the exchange of knowledge pertaining to the construction and management of water works. From an original membership of only TWENTY-SEVEN, its growth has prospered until now it includes the names of 900 men. Its membership is divided into six classes, viz.:

A Member shall be an officer or employee of a public or private water works, an engineer, chemist or other person qualified to aid or interested in the advancement of knowledge relative to water works.

An Honorary Member shall be a person of acknowledged eminence in some branch of water supply or of engineering.

A Junior shall be not less than eighteen years nor more than twenty-five years of age, a student or connected with water supply work.

An Associate shall be either a person, firm or corporation engaged in manufacturing or furnishing materials or supplies for the construction or maintenance of water works.

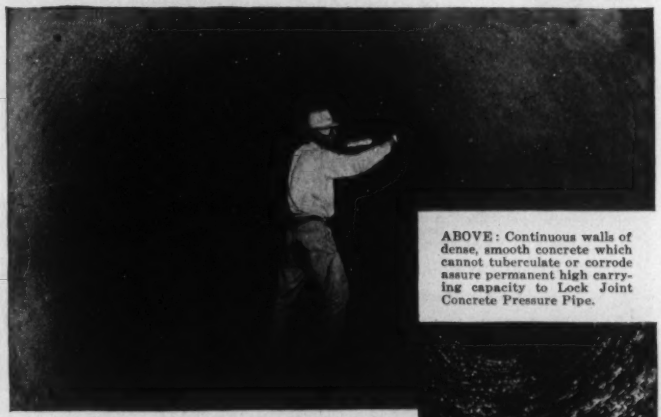
A Corporate Member shall be either a Water Board, Commission, Company or Municipal Corporation.

The initiation fees and annual dues are as follows:

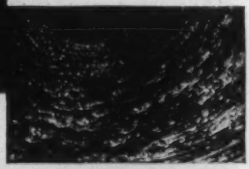
	Initiation Fees		*Annual Dues*
Members	$ 5.00	Members	$ 5.00
Juniors	1.00	Juniors	1.00
Associates	10.00	Associates	20.00
Corporate Members	10.00	Corporate Members	10.00

This Association has at least eight regular meetings each year, of which five are held in Boston, one in northern New England, one in southern New England, and one, the annual convention, held in September or October on such date as the Executive Committee may designate.

CONCRETE *is the answer to your main problem*



ABOVE: Continuous walls of dense, smooth concrete which cannot tuberculate or corrode assure permanent high carrying capacity to Lock Joint Concrete Pressure Pipe.



ABOVE: The tuberculated condition of this metallic pipe reduces its carrying capacity 50%.

Tuberculated mains—corroded mains—leaky mains—all cost municipalities, taxpayers and industry immense sums every year in added pumping costs, maintenance and repairs. Yet, by the use of Lock Joint Concrete Pressure Pipe, all of this unnecessary expense could be eliminated.

1. LOCK JOINT PRESSURE PIPE does not tuberculate...

RESULT
- **No excessive pumping costs**
- **No periodic cleaning costs**
- **No loss of income from inadequate delivery of water.**

2. LOCK JOINT PRESSURE PIPE does not corrode...

RESULT
- **No cost for major repairs**
- **No cost for periodic patchwork**
- **No loss of revenue while the line is closed for repair.**

Consider these facts when you plan your next water supply or transmission main and specify Lock Joint—the pressure pipe of perpetual economy.

LOCK JOINT PIPE COMPANY

Est. 1905

P.O. Box 269, East Orange, N. J.

Pressure Pipe Plants: Wharton, N. J.
Turner, Kan. • Detroit, Mich. • Columbia, S. C.

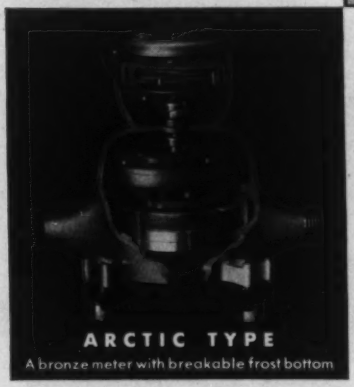
BRANCH OFFICES: Casper, Wyo. • Cheyenne, Wyo. • Denver, Col. • Kansas City, Mo. • Valley Park, Mo. • Chicago, Ill. Rock Island, Ill. • Wichita, Kan. • Kenilworth, N. J. • Hartford, Conn. • Tucumcari, N. Mex. • Oklahoma City, Okla. • Tulsa, Okla.

Lock Joint Pipe Company for over forty years has specialized exclusively in the manufacture of reinforced concrete pipe for water supply and transmission mains as well as for sewers, culverts and subaqueous installations.

How you benefit from the *STAINLESS STEEL TRIM* in all Rockwell Disc Meters

SURE, stainless steel costs more than brass. But we use it freely* in Rockwell disc meters for the corrosion resistance and durability it alone provides. We can afford to offer this extra value only because of our huge production volume. We built this volume by building a better product. Now we can pass on the benefits to you in the form of even superior performing meters —meters that last longer because of this construction and are easy and economical to repair.

TROPIC TYPE
All-bronze, split case construction



ARCTIC TYPE
A bronze meter with breakable frost bottom

All these Vital Parts Are Made of Stainless Steel:
division plates, driving dogs, disc and stuffing box spindles, screws.

Write for bulletin

ROCKWELL
MANUFACTURING COMPANY
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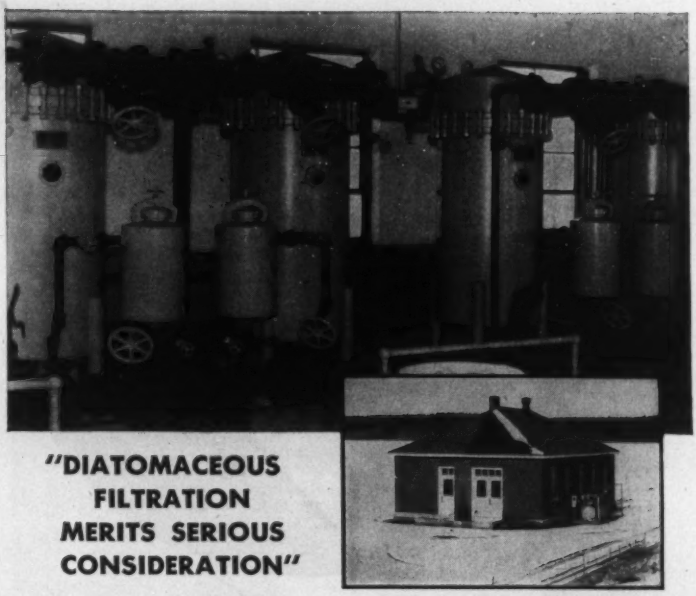
The Symbol for Service, Quality and Performance in Water Meters



"DIATOMACEOUS FILTRATION MERITS SERIOUS CONSIDERATION"

In the February 1954 issue of the Journal of American Water Works Association, Mr. J. K. Fraser (Civ. Engr., Barker & Wheeler, Engrs., Albany, N. Y.) states "Where intermittent filtration must be provided and where the water is of a character that lends itself to filtration by diatomaceous earth, such a process MERITS SERIOUS CONSIDERATION."

The photo shows a diatomaceous earth filtration system installed at Tupper Lake, N. Y. In his article, Mr. Fraser tells why this 1.75 MGD system was selected in preference to a sand filter system. He also speaks about the flexibility of the system — about its quick starting characteristics — about its ease of backwashing and economy of operation.

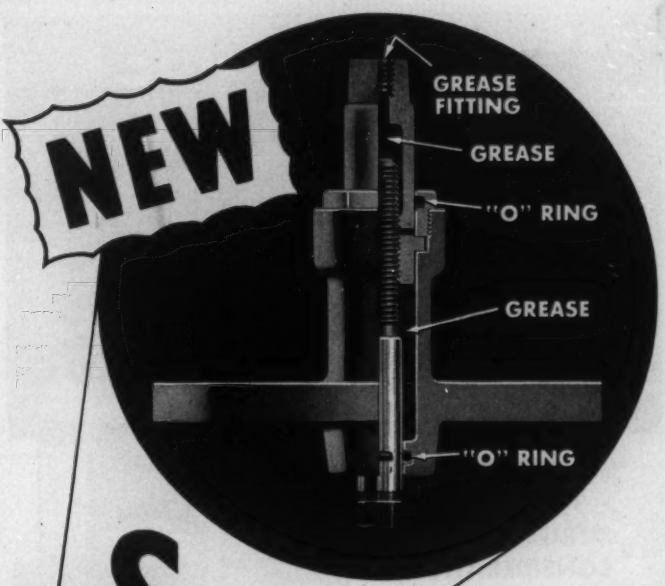
We will be happy to send you a copy of Mr. Fraser's article "Diatomaceous-Earth Filtration in New York State" — also Bulletin 1800 describing Proportioneers Purecel Diatomite Filtration Systems. Proportioneers, Inc., 366 Harris Ave., Providence 1, R. I. Technical service representatives in principal cities of the United States, Canada, Mexico and foreign countries.

% PROPORTIONEERS

DIVISION OF B-I-F INDUSTRIES, INC.

BUILDERS IRON FOUNDRY • OMEGA MACHINE CO. • BUILDERS-PROVIDENCE, INC.

BIF METERS FEEDERS CONTROLS

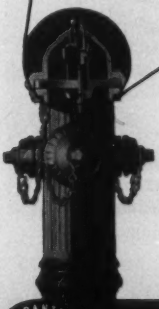


47

SMITH HYDRANTS

WITH "O" RING SEAL AND GREASE CHAMBER FOR OPERATING THREADS

When you specify "O" Ring Seals rather than conventional packing you reduce maintenance to a minimum. The lower "O" Ring in the pressure seal, the upper "O" Ring the dirt seal. The specially compounded rubber plastic "O" Rings insure a long life pressure seal. Each time the hydrant is operated, the "O" Ring Seals, the bearing surfaces, and the operating threads are automatically lubricated. The lubricant is suitable for temperatures of minus 30 degrees to 200 degrees. All Smith Hydrants including hydrants in service can be fitted with "O" Ring Seal Plates.

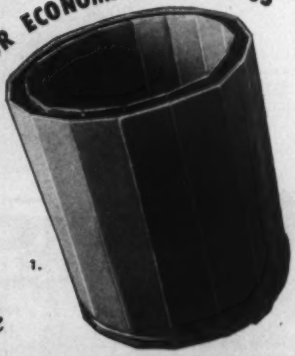


THE A.P. SMITH MFG. CO.

EAST ORANGE, NEW JERSEY

1. Large illustration shows Singl-use Fiberform ready to be filled.
2. Filling plate and collar simplify pouring concrete into form.
3. Completed meter box barrel.
4. Meter box with cover in place.

FOR ECONOMICAL SETTINGS



Make Your Own

CONCRETE METER BOX BARRELS *with* FORD *Singl-use Fiberforms*

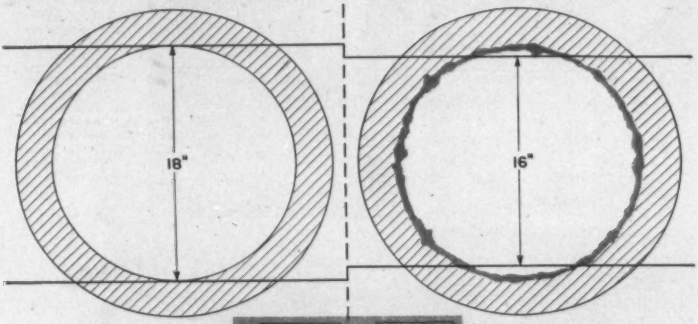
Water utilities that find vitrified or concrete tile expensive or difficult to obtain, may now make their own barrels for outside meter settings with Ford Singl-use Fiberforms which are inexpensive and easy to use. Forms made for 15", 18" and 20" ID barrels and in lengths of 15", 18" and 24". Use two for deep settings. Send for full information.

FOR BETTER WATER SERVICES

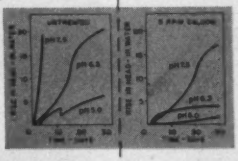
FORD

THE FORD METER BOX COMPANY, INC. Wabash, Indiana

CORROSION CONTROL FOR WATER MAINS



Tuberculation not only results in permanent damage to the pipe, but restricts carrying capacity drastically.



In this case, flow in this 18" pipe was reduced from 4.8 mgd to 3.3 mgd by tuberculation within several months. The effective diameter of the 18" pipe was therefore reduced to only 16".

Calgon*

greatly reduces tuberculation
. . . more than pays for itself

A rapidly growing eastern municipality found that, due to tuberculation, the 18" main carrying their principal raw water supply had to be cleaned as often as three times a year. After each cleaning, despite treatment with lime and chlorine, the restored flow dropped off rapidly from 4.8 mgd to 3.3 mgd, at which point the main had to be cleaned again.

Then Corrosion Control with 3.0 to 3.5 ppm of Calgon was instituted, and rate of flow held up so well that now the main is cleaned only once a year.

A substantial reduction in cleaning costs, but this saving is only part of the story. In addition, this municipality has realized a saving of over $40 per day in pumping costs, and no red water complaints have been received since Calgon treatment was instituted.

Corrosion Control with Calgon† is effective against tuberculation, and stabilizes dissolved iron and manganese. Corrosion Control with Calgon† is simple, effective, *and* economical. Let us tell you how Calgon can help you with *your* water problem.

HAGAN HALL BUROMIN CALGON

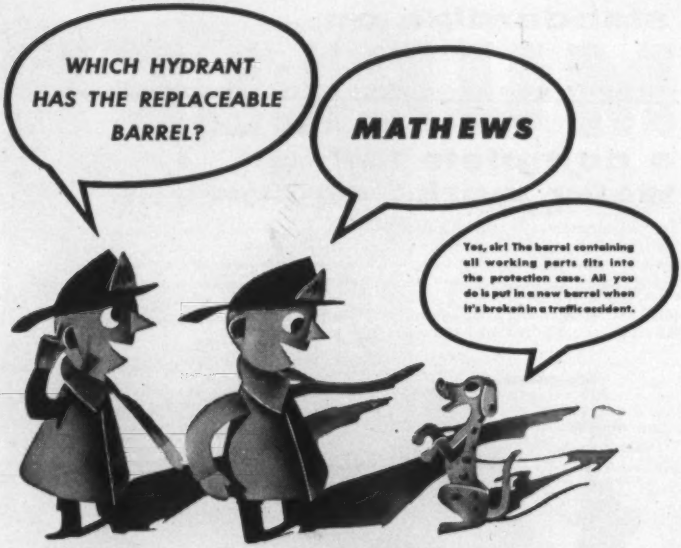
CALGON inc.

A SUBSIDIARY OF HAGAN CORPORATION

HAGAN BUILDING, PITTSBURGH 30, PA.

*Calgon is the registered trade mark of Calgon, Inc. for its vitreous phosphate products.

†Licensed under U.S. Pat. No. 2,337,856.


WHICH HYDRANT HAS THE REPLACEABLE BARREL?
MATHEWS
Yes, sir! The barrel containing all working parts fits into the protection case. All you do is put in a new barrel when it's broken in a traffic accident.

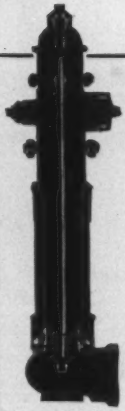
MATHEWS MEANS FIRE PROTECTION

LONG LIFE. Mathews Hydrants are protected at wear points by bronze bushings — corrosion and rust are licked.

TROUBLE-FREE. The stuffing box plate, cast integral with the nozzle section, provides a positively leakproof construction — water, sediment and ice cannot interfere with operating thread.

LOW-COST MAINTENANCE. Only one point requires lubrication — and that only during routine inspections.

REPLACEABILITY. A Mathews leveled in an accident can be replaced without excavating and in a jiffy. Community protection is interrupted only for minutes.



The Mathews includes every quality feature...

Compression type valve to prevent flooding • Head turns 360° simply by loosening bolts • Replaceable head • Nozzle sections easily changed • Protection case of "Sand-Spun" cast iron for strength, toughness, elasticity • A modern barrel makes an old Mathews good as new • Mechanical-joint pipe connections as needed

MATHEWS HYDRANTS

Made by R. D. Wood Company

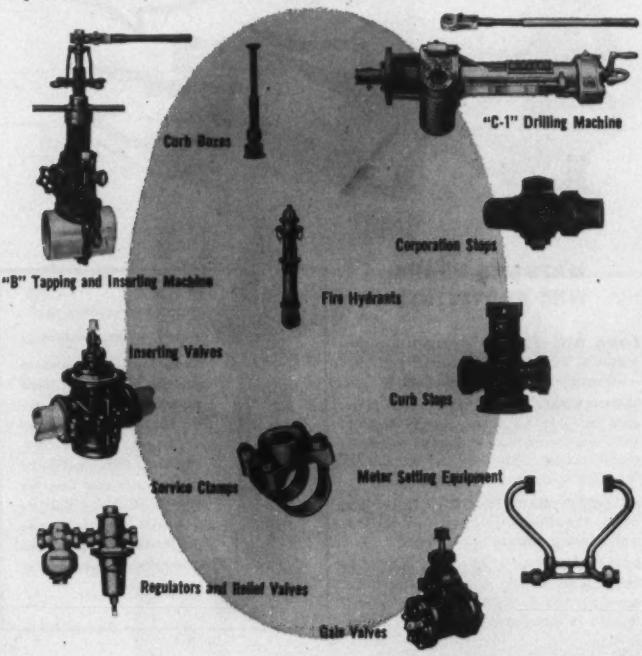
Public Ledger Building, Independence Square, Philadelphia 5, Pa.

Manufacturers of "Sand-Spun" Pipe (centrifugally cast in sand molds) and R. D. Wood Gate Valves

standardize on MUELLER

a complete line of water works equipment

Low maintenance and high efficiency are characteristic of all Mueller water works equipment, supplies and specialities. Each item is engineered to do its specific job and is manufactured from the finest of materials — guaranteeing long, trouble-free life. A high degree of standardization and interchangeability requires a minimum inventory of replacement parts in case of accident or emergency. Mueller's complete line of products, a few of which are shown, provides a single source for your water works needs. All products are fully tested and warranted.



MUELLER CO.

Dependable Since 1857

MAIN OFFICE & FACTORY DECATUR, ILLINOIS

water
is important to everybody...

The Miss at this fountain of youth would be mighty disappointed if low water pressure were suddenly to deprive her of a drink—and so would millions of others who take water pressure for granted. One of the best, and least expensive ways to assure the constant flow of water is through Centrilining.

The Centriline process thoroughly cleans and centrifugally applies cement-mortar to the interior of pipe lines in place—prevents corrosion, tuberculation and leakage; minimizes interruption of line operation and street traffic. For an effective, economical means of increasing flow capacity, look to Centriline.

Write for free booklet today.

CEMENT-MORTAR LININGS FOR PIPES IN PLACE

3,000,000 FEET **OF EXPERIENCE**

CENTRILINE CORPORATION

A subsidiary of Raymond Concrete Pile Co.

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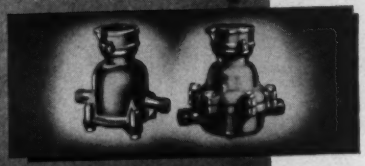
Branch Offices in Principal Cities of United States and Latin America

WATER
is vital...



BADGER meters conserve it!

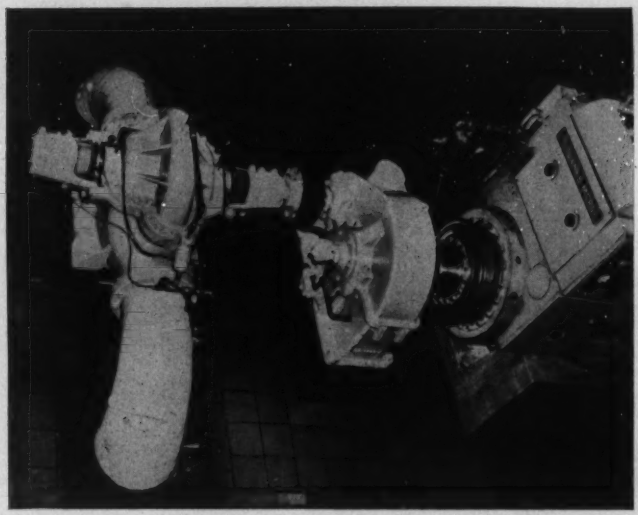
More than 25 million acres of crops must be irrigated to provide America's food supply. The Badger water meters used in thousands of communities throughout the nation help save water for this vital need . . . measure water accurately for homes and industry . . . provide a precise check on water usage and waste. With their outstanding record of performance, Badger meters have won the praise of waterworks men everywhere . . . are known as the meters that save water, work, money!

BADGER WATER METERS

BADGER METER MFG. CO., Milwaukee 45, Wisconsin

"Measuring the water of the world"

ANOTHER FAIRBANKS-MORSE LEADER



12"x14" Fairbanks, Morse Figure 5814 Pumps

Pictured above is a bird's eye view of one of the two 12" x 14" Fairbanks, Morse Figure 5814 Double Suction Horizontal Split Case Centrifugal Pumps, supplying the Arlington, Arlington Heights and Belmont sections of the Metropolitan District Commission's Water Division.

Each of these two pumps rated at 6000 G.P.M. at 220 TDH is driven by an eight cylinder Model 31A8½ Fairbanks, Morse Diesel Engine.

Installed in early 1953 as part of the MDC's new expansion program, both pumps take suction from the Wachusett Reservoir, Clinton, Massachusetts, distributing main and boost the pressure in the area served.

A trained Fairbanks, Morse Engineer is on hand to help solve YOUR pumping problems.

Write or telephone to:

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Need water pipe fittings? Call us at Builders-Providence — in most instances we have the right fitting in stock, ready for immediate shipment to you. Remember, we are as close as your nearest phone . . . don't hesitate to call. Builders-Providence, Inc. (Division of B-I-F Industries, Inc.) 366 Harris Ave., Providence 1, Rhode Island.

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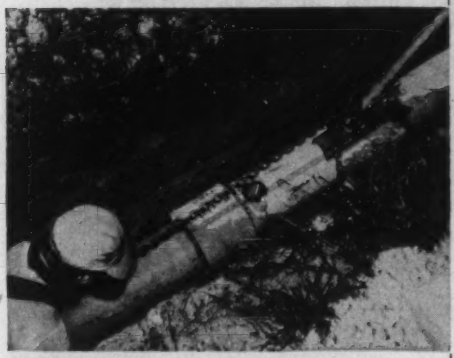
DIVISION OF B-I-F INDUSTRIES, INC.
BUILDERS IRON FOUNDRY • PROPORTIONEERS, INC. • OMEGA MACHINE CO.
BIF METERS FEEDERS CONTROLS

Save time and money on water line installation



In this Georgia installation, faster line assembly permitted following the digger closely.

On this Utah job, snow, ice and frozen ground caused no installation slow-down.

Transite® Pressure Pipe with the new Ring-Tite® Coupling

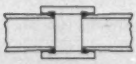
Goes together fast! Locks tight automatically!

In state after state, community after community, Transite Pressure Pipe and the new Ring-Tite Coupling have made possible important cost-savings in installation time.

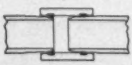
The Ring-Tite Coupling is effecting installation savings everywhere because of its special features. To begin with, pipes are aligned quickly, easily. Coupling design not only provides automatic aligning but also automatic adjustment for expansion. Rubber rings are simply popped into grooves for assembly; lubricated pipe ends then slide in under rings smoothly, easily, and surely. Pipe ends are automatically separated within the coupling. This separation gives the line flexibility to withstand shock and vibration, relieves line stresses, permits conformance to curves.

Like Transite Pressure Pipe, the Ring-Tite Coupling sleeve is made of asbestos and cement . . . it is strong and durable, cannot rust, and is highly resistant to corrosion. Installations can be completed under adverse weather, temperature or terrain conditions. No complicated equipment is required.

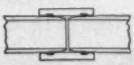
For more information write Johns-Manville, Box 60, New York 16, N. Y.



A. Rubber rings popped into grooves before assembly.



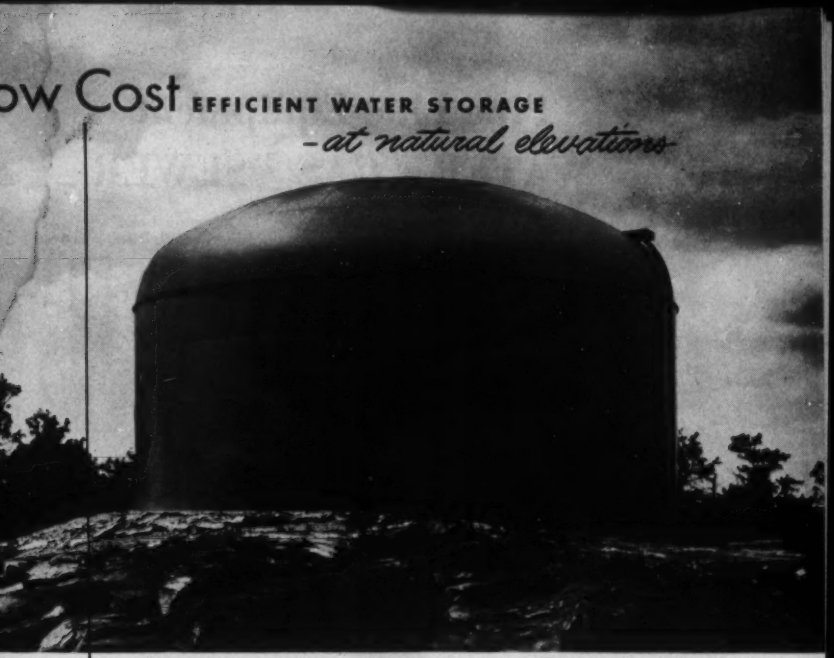
B. Lubricated pipe slides in under rubber rings.



C. Both pipes in assembled position. Rings compressed and locked in place.

Johns-Manville TRANSITE PRESSURE PIPE

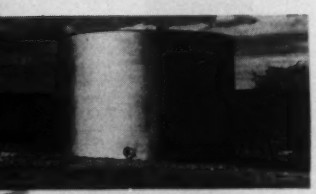
AN ASBESTOS-CEMENT PRODUCT



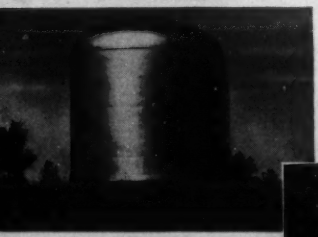
Worcester, Mass.—Ellipsoidal roof type, 1,500,000 gallons capacity.

* Steel Reservoirs

by PITTSBURGH • DES MOINES



Liberty, N.Y.—Flat roof type, 300,000 gallons capacity.



St. Paul, Minn.—Ellipsoidal roof type, 2,000,000 gallons capacity.

The advantages of strength, simplicity, and economy of construction make Pittsburgh-Des Moines Steel Reservoirs your outstanding choice for water storage where elevated sites are available.

Durable for a lifetime, guaranteed in design, workmanship and material, P-DM Steel Reservoirs are built in flat-, dome-, or ellipsoidal-roof types, unornamented, or with pilasters and other architectural features as specified. *Write for our latest descriptive Steel Reservoir Bulletin.*

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Pipe Line Equipment
WATER · GAS · SEWER

Bottled Gas Lead Melting Furnace

Calking Tool Outfit

For water mains up to 12". Strong leather bag. Keeps the right tool for every job within easy reach.

Portable Lead Melting Furnace

A handy outfit for quick action, equipped with gasoline burner.

Asbestos Joint Runners

Best quality asbestos rope, brass caps attached to each end. Equipped with clamps.

Pipe Cutter

Inexpensive way of cutting pipe. Every wheel in contact with the pipe is a cutting disc, so cutter need be moved only a short distance to cut entirely around the pipe.

M-D Cut-In Connections for Sewers

A light cast iron fitting with bell end for connecting house service with main sewer pipe without necessity for placing a Y-branch or T-branch in the sewer line.

Test Plugs for Bell or Spigot End Cast Iron Pipe.

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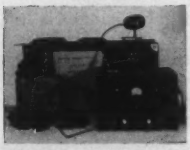

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CHLORINATION—A TEN-YEAR REVIEW

BY A. E. GRIFFIN*

[*Read January 22, 1953.*]

"Chlorination turned a corner," according to an authority of national repute (1), in 1939. This has since proven to be a true prophecy and there is now every indication that chlorination is on the "straightaway." This is borne out by the fact that the principles advanced in the early 1940's, relating to "break-point chlorination" in particular, have been confirmed, strengthened and expanded by subsequent research and practice. In 1944, at the time of an earlier review (2) of chlorination, there was considerable confusion as to the meaning of some of the terms being used and even as to accuracy of some of the data being presented. Such weaknesses have now largely been remedied because enough time has elapsed to allow a larger number of people to study, digest and put in practice the principles involved in the then new concept of chlorination (3).

Terminology

Faber (4) reported in 1946 that a committee of the American Water Works Association had proposed a unified terminology concerning chlorine application, which was subsequently accepted as official. The adoption of these terms marked an important step in the clarification of the issue, largely because chlorination was reduced to two types: namely, *free residual chlorination* and *combined residual chlorination*. This has had the effect of greatly facilitating both plant practice and research.

Speed of Kill

Butterfield, Wattie, Megregian and Chambers (5) in 1943 pointed out that (a) the time of exposure of bacteria to free chlorine is a primary factor governing the extent of the bacterial kill; (b)

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hydrogen-ion concentration has a marked effect on the bactericidal efficiency of free chlorine, the killing power diminishing with increasing pH values, and (c) increase in temperature tends to increase the bactericidal properties of free chlorine. This knowledge should be very beneficial where extreme temperatures and pH values are encountered. One cannot expect, for instance, that the action of chlorine at a pH of 9.5 and a temperature of 39° F. would be as rapid as a similar residual at pH 7.0 and a temperature of 70° F.*

Similarly, Butterfield and Wattie (6) showed that the length of exposure of bacteria in water to chloramine and amount of chloramine present are primary factors governing rate of bacteriological kill. Under favorable condition of 7.0 pH and 20° to 25°C., 100% kills cannot be expected in less than 20 min with a chloramine concentration of approximately 1.2 ppm. The rate of kill varies directly with the temperature. Chloramine residuals are much less efficient than the free available chlorine residuals and, to effect 100% kills with same exposure, about 25 times as much chloramine as free chlorine is required. If the same kills with the same amounts of residual are to be obtained, then the exposure to chloramines must be 100 times that to free chlorine. Both of these reports were condensed by Butterfield (7) into a short article in 1948 as a guide toward proper chlorination. Kabler (8) and Kabler, Pierce and Michaelson (9) indicate that "the proposed minimum safe chlorine residuals are about twice the average amount required to kill the bacterial species examined and are under all conditions of test appreciably greater than the amount of available chlorine required to kill the most resistant species."

Testing for Chlorine Residual

The ortho-tolidine-arsenite test, first proposed by Hallinan (10), proved so successful that it has been quite generally adopted by the water-works profession. Gilcreas and Hallinan (11) used it in the field, showing that at that time very little free chlorine was present in the majority of waters in New York State. Gradually this method has given way, particularly from a precision standpoint, to the amperometric titration method, as described originally by Marks and

* This JOURNAL, 1944 — see (2).

Glass (12) in 1942. This is a method utilizing an electrical end-point. This method has now been improved to the point where total chlorine, total chloramine, and mono- and di-chloramine can be determined very accurately (13).

Prior to the disclosure that mono- and di-chloramines could be detected by means of the amperometric method, Palin (14) (15) published a method involving a series of titrations with standard ferrous ammonium sulfate solution, using the blue meriquinone color of neutral ortho-tolidine hydrochloride as an indicator. This method was reviewed by Marks, Williams and Glasgow (16) and compared with the amperometric method. The implication of the conclusion is that the latter method is somewhat more precise than the former.

Contemporary work along these same lines includes Connell's (17) ortho-tolidine titrating method and Taras' (18) micro-titration method. Chang (19) deviated from the beaten path and restudied the oxidation-reduction concept of chlorine residuals without arriving at a specific relationship. He did, however, introduce the concept of dissociation of HOCl in water at varying pH values. This then was carried to workable conclusions by Fair (20) (21) and Moore (22).

Enzyme Theory of Bacterial Death

The enzyme theory of bacterial destruction with chlorine was developed during this same period by Green and Stumpf (23).

Effect of Chlorine on Virus

Although most public health officials are primarily interested in the removal of coliforms and common pathogens, other investigations have concentrated upon the effect of chlorine upon the more obscure, possibly water-borne disease, poliomyelitis. Krumbiegel (24) has pointed out that there is at present no epidemiological evidence that permits the belief that water is of any importance in the spread of poliomyelitis. Armstrong (25) has claimed that a residual of 4.0 ppm chlorine with a contact of 24 hours will inactivate the virus of poliomyelitis in slightly turbid, artificially contaminated water and that a residual of 0.4 ppm will suffice in clear water. This evidence has been largely substantiated by Lensen (26) (27) (28) and Ridenour (29).

Chlorine Demand

Chlorine demand is usually defined as the difference between the amount of chlorine applied and the chlorine residual remaining at any given period of contact. The information thus obtained has been generally used in estimation of chlorinator capacity, but Feben (30) (32) and Taras (31) report that it can be used in plant control. Together they have concluded, after a long series of experiments and plant operation, that there is a mathematical relationship between the chlorine demand at one contact point and that at any other point, provided free available chlorine is present. These relationships make it possible to maintain specific residuals in the finished water. Furthermore, they claim the constants obtained offer a clue as to the nature of the chlorine-demand substances present in a natural water. A nomograph for rapid determination of the chlorine demand at stipulated intervals has been evolved. It is quite possible that these relationships would be invaluable in case of sudden gross pollution, such as from war gases. Faber (33) in 1942 hinted that any sudden, sharp increase in chlorine demand should be investigated, because it could be an index of pollution. Now, with the Feben-Taras formula any such pollution would be instantly detected.

Main Sterilization

Bacterial pollution of newly laid mains, in the case of natural breaks or as a result of some disaster, is not uncommon and, before the mains are placed in service, they must be sterilized. According to the record, this may be accomplished either by heavily chlorinating the water as it fills the main and then allowing the section to remain static and out of service for a period, or by continuously chlorinating the water to a medium residual (0.5 to 2.0 ppm) over a period of a week or more, the water, meanwhile, being used for normal purposes. Billings (34) advocates the former method, as does the American Water Works Association Procedure for Disinfecting Water Mains, 7D.2—1947, approved Sept. 30, 1947 (35). Calvert (36), on the other hand, is neutral and proposes that either may be used as the situation dictates. Heller (37) (38) is the main advocate of the continuous-flow method and claims to have obtained signal success thereby.

Tank Disinfection

For the disinfection of tanks and reservoirs, Tracy (39) and Tracy and Fonseca (40) recommend that chlorine spray be used. They aver that chlorine solutions as strong as 500 ppm can be used without difficulty.

Emergency Chlorination

Emergency chlorination always requires considerable advance planning. That such is being done is evidenced by the equipment used by Moore (41), entailing truck-mounted equipment, which includes pumps, hypochlorinators, containers for the "bleach" and the necessary hose and tools. To facilitate the work he has prepared a chlorine-dosage chart for various size and lengths of mains. Sopp (42) likewise has outlined his plans for emergency. It is known that many other cities have similar or even more elaborate systems that have never been reported in the literature.

Chlorine Handling

The safe handling of chlorine is a perennial subject and an interesting one. Every operator should have a copy of the Chlorine Manual (43) in his possession with the data therein. Lordley (44) has an ideal set-up at Richmond, Va. Miller (45) at Norfolk, Va., transports one-ton cylinders on a specially built truck, which carries its own skids and winch. Barbi (46) has described a method of outside control for all motors in the chlorinator room.

Chlorine Leaks

What to do in case of a chlorine leak is always a moot question. Hopkins (47) has answered the question as related to water by a plant-scale demonstration. In collaboration with the fire department water spray, called fog-nozzle spray, was directed against liquid chlorine on macadam road and against chlorine gas in the air. It was concluded that the gas can be dispersed by such a method, but the gas cannot be absorbed. The method is no substitute for permanent ventilation.

Local Production of Chlorine

Black and Hoskinson (48) during 1950 investigated the possibility of local production of chlorine. Chlorine-consumption forecasts were presented, together with data on generation costs and economies, as related to Florida particularly. Sacramento experiences were cited.

Chlorine-Residual Recording

The introduction of the chlorine-residual recorder is probably the outstanding advance made in chlorination during the decade under consideration. The instrument consists of two main units, a sampling cell and recorder. The sampling cell contains two metallic electrodes, together with a grit-bombardment system, which abrades dirt and other foreign substances from the electrode. As the sample flows past the electrodes to waste, a direct current is generated, proportional to the free available chlorine residual present in the sample. This current flows to the recorder mechanism, where it is interpreted in terms of free chlorine residual or is transmitted to a remote indicator. Baylis (49) has reported on the operation of the early type as installed at Chicago. Hazey (50) tells how it has smoothed out the final residual content at a plant where chlorine demand fluctuates over wide ranges and Krum (51) has told us that it has reduced the incidence of chlorinous tastes, due to periodic overdosing with chlorine during short shut-down periods. Normally it is difficult continuously to record residuals in a settling basin, because of floc interference. Breneman, Rollings and Ulrich (52) solved this problem by using a "bottomless clarifier". This consisted of nothing more than an inverted can, suspended in the basin, vented to prevent air binding. The sample, obtained from under this can, is floc-free and flows unimpeded through the sample cell. Weir (53) has described the portable amperometric residual recorder.

No decade is complete without the description of home-made equipment of one kind or another. This decade is no exception. Caldwell (54) is using an automatic residual recorder, built by himself, patterned after the Montreal design using the blue color range of ortho-tolidine. Gray and Moses (55) also use what they call a chlorine analyzer, based on the photo-electric cell principle on the yellow range of ortho-tolidine. Both appear to be functioning well,

but neither has been placed on a commercial basis. Barbee (56) on the west coast adapted a complicated program disc regulator, self-built, to his installed chlorinator. There is no record of any other such installation. Mahan (57) avers that it is not difficult to construct an amperometric titrator, and presents data and pictures to show how it is done.

Taste and Odor Control

The one big remaining problem in water purification is taste and odor control. The tastes and odors are due to a variety of causes, some known and others unknown. Among the unknowns have been the persisting musty, woody tastes and odors that come and go, seemingly in spite of everything. They have sometimes been termed the "melting-snow taste and odor", but that does not account for the same tastes and odors in the warmer climates. Silvey (58) now attributes these difficulties to *actinomycetes*. He claims these organisms grow in the reservoirs and can only be killed by treatment with chlorine dioxide. Experimental work on this is continuing.

Phenolic tastes and odors are more common and can be effectively controlled by applying approximately 0.5 ppm chlorine dioxide, according to Synan (59), and with chlorine, according to the equations written by Ingols (60). Ettinger (61) presents data likewise that free residual chlorination will oxidize phenolic compounds and produce a taste- and odor-free water. He also suggests that stepwise chlorination is especially effective.

Adams (62) claims that free residual chlorination will rid water of most industrial-waste odors; Williams (63) that it will rid water of organic wastes, and Riddick (64) claims in addition that maintenance of free residuals on the order of 3.5 ppm, followed by dechlorination, will produce a tasteless water and that the color will be lowered.

Chlorine Dioxide

The preparation and uses of chlorine dioxide have been discussed by Synan (65), Vincent (66) and Aston (67) (68). The bactericidal properties of chlorine dioxide have been investigated by Ridenour (69), Ingols (70) (71) and McCarthy (72). All seem to agree within limits that chlorine dioxide is about equal to chlorine in

a water in which the chlorine demand is satisfied, and somewhat better than chlorine at high pH values.

Chlorine dioxide has been demonstrated to be especially effective in the elimination of chloro-phenolic tastes and odors. This is borne out by the experiences of Mounsey and Hagar (73), Aston (74) and Coote (75). Faber (76) thinks this is because the chlorine dioxide forms a tasteless compound of phenol, perhaps the trichloro-compound.

Break-Point Process Plant Experiences

When the break-point process was first introduced, it was considered by some as a "chemical curiosity", but this attitude changed rapidly as the effectiveness of free available chlorine residuals was revealed both in the laboratory and in plant practice. This is attested by the papers that have been presented at the various water-works meetings throughout the country during the past ten years.

Chandler (77) at Beckley, W. Va., for instance, has reported that use of the process there effected a saving of 6 gpg of alum and, in addition, reduced the color, iron and manganese content much lower than was ever possible without chlorinating to and beyond the break-point. The chlorine dosages, to obtain such results, varied between 4.5 ppm and 12.0 ppm. At New Albany, Ind., Adams (78) found that the maintenance of free residuals reduced the gas-forming bacterial content far below the point ever attained with heretofore normal chlorination practices. Jacobson and Wellington (79) have documented the comment that the use of free residuals at New Haven "has permitted a more scientific method of control, whereby more uniform and improved taste and odor control, as well as bacteriological results, is obtained". Almquist (80) has given the same impression regarding other supplies in Connecticut. At Plattsburg, N. Y., Behan (81), through the use of free residual chlorination, as high as 3.5 ppm in the filter effluent, has been able to omit the use of alum during the greater part of the year. This was because the use of high amounts of chlorine bleached the color to an acceptable figure, from 30 to less than 10, and produced an essentially bacteria-free water.

Coote (82) reported that the use of free residuals at Valparaiso,

Ind., improved disinfection as well as the palatability of the water. At the same time Crabill (82) said that free residuals improved taste and odor control, improved coagulation and eliminated slime growths from the sand in the filters. Gard (83) found that if a six-hour contact with chlorine were provided ahead of coagulation, tastes and odors in the water system at Greenhaven, N. Y., were eliminated. Shorter contact periods were not effective. To obtain such results, 0.6 to 0.7 ppm chlorine residuals were maintained in the distribution system. Horace Brown (83a) again in 1950, as in 1939, seemed to go one step farther than anyone else. In 1939 he won the prize by successfully applying 110 ppm chlorine to produce an acceptable and safe water. In 1950 he applied chlorine at three points within the plant before the water was released to the distribution system. A summary of comparative plant bacteriological results, before and after maintenance of free residuals, was presented to the American Public Health Association in 1945 by Griffin and Chamberlin (84). The tabulated data were dramatic, in that wherever the break-point process was used, the bacterial content consistently approached the vanishing point.

Control of Nuisance Organisms

With all the evidence indicating that free residuals kill practically all types of bacteria, it was perhaps no surprise to water-works officials in general that the maintenance of such residuals also killed slime-forming bacteria, wherever they might grow. Slime growths are important in that they interfere with the flow of water, cause tastes and odors, accelerate corrosion and are otherwise offensive. Thus they are sometimes known as nuisance organisms. It thus follows that an eternal warfare is waged against slime. Rogers (84a) has reported their removal from the transmission lines at Wichita, Kan., with chlorine, although as a main-cleaning job it was not a success. But no one has ever claimed that chlorination takes the place of mechanical main-cleaning. Griswold (84b) has had a long record of signal success —1933 to date—at Utica, N. Y., in maintaining line capacity by using the chlorine-ammonia process. His observations are substantiated by carefully collected data over a period of years. On the other hand, Jackson and Mayhan (85) at Little Rock, Ark., prefer to

use chlorine alone for this purpose, as does Derby (86) in California.

Wilson, Brown, Pomeroy and Montgomery (87) have experienced much difficulty with sulfur and iron bacteria in a distribution system for which they are responsible. Such difficulties have been largely overcome by judicious use of chlorine.

Control of Organisms in Distribution Systems

Reed (88) in Boston recognizes that many types of bacteria, other than pathogens, exist in the Metropolitan System and is studying methods for their control.

Levine, Heller and Bender (89) have reported that free residuals are more dependable for bacterial control in the distribution system than chlorination to a stipulated residual without regard to type.

Miscellaneous Uses of Chlorine

Other than a killer of bacteria, chlorine has many other attributes, as reported during the period under consideration. It may be used to remove iron and manganese, as attested by Mathews (90) and Edwards and McCall (91); to increase the capacity of slow-sand filters, as demonstrated by Jacobson and Wellington (92); to restore well capacity, as shown by Brown (93) and Huston (94), or to control bacterial growth in zeolite softening material, as pointed out by Klumb, Marks and Wilson (95).

Super- and de-chlorination continues to be successfully used at Toronto (96) and has recently been adopted at Washington, D. C. (97).

The chlorine-ammonia process is still a valuable tool in water treatment, as attested by Fuller (98) (99). Smith (100) has reported that the use of copper sulfate is a valuable adjunct to both chlorine and ammonia.

Finally, to chlorinate properly a water containing abnormal amounts of albuminoid ammonia, Williams (101) (102) has presented a wealth of data to the effect that the ratio of mono- to di-chloramines should be carefully watched.

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PROGRESS OF THE WALTHAM WATER WORKS

BY CLARENCE B. FRENCH*

[*Read February 19, 1953.*]

The Waltham Water Works were started under Chapter 337, Acts of 1872, which allowed Waltham to take water from the Charles River or Stony Brook Valley, not to exceed 1.5 mgd. The Engineer's report of the various sources of supply is summarized as follows:

Sandy Pond, Lincoln, not sufficient quantity.

Sudbury River, quantity but not quality.

Hardy's Pond, Waltham, not sufficient quantity.

Stony Brook, sufficient quantity but afraid of the mill privileges, which might prove costly.

Charles River, quantity and quality satisfactory; the most economical source.

As a result, the Charles River source of supply was finally accepted, and it was recommended that water be pumped from a basin to be constructed about 100 ft. from the river, so that there would be natural filtration; also that two pumping engines be erected, of 500,000-gal. capacity in 10 hours, to raise the water to a reservoir, the location of which was to be selected later. This location, next to the Charles River, resulted from several civil suits from the mill owners and others, which were not completely settled until 1900.

The first report of the Board of Water Commissioners, dated June 1, 1874, showed that during the construction of the basin from which to pump, a large quantity of water boiled from the land side of the basin at the rate of 3 to 4 mgd. This was very encouraging.

Cement-lined wrought-iron pipe was decided upon for the distribution system, because of the high cost of cast-iron pipe. The 12-in. force main, however, was cast-iron and was bought from the Warren Foundry.

Water from the basin flowed by gravity through a stone conduit

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to a pump well, located in the basement directly below the pump. From there it was pumped to an open reservoir about 2,700 ft. away, constructed on a hill at an elevation of about 206 ft. above sea level. This reservoir had a storage capacity of some 6 mil. gal. From the reservoir water flowed by gravity throughout the distribution system.

Only one 1¼-mgd. Worthington pump was purchased at first.

In 1876 the works were substantially completed at a cost of about $261,000. They consisted of about 18 miles of distribution mains, with sizes from 4" to 16", and about 900 services and 103 hydrants. Incidentally, the hydrant rental at that time was $25.00 per hydrant, whereas now it is only $5.00. The daily average consumption was about 481,000 gal. per day. Cement-lined wrought-iron service pipe was adopted and has been used ever since.

As early as 1878 considerable trouble was being experienced in the cement mains, due to breaks caused by water hammer, resulting in the closing of hydrants. The result was that the Boston hydrant was condemned and the Chapman hydrant adopted. A year later the cement pipe was also condemned and cast-iron pipe was accepted for all mains. This was after several breaks during thunder showers —particularly one which destroyed over 400 ft. of main.

In 1880, the installation of a second pumping engine insured the city of better protection.

Six years later the water-works reservation was considerably enlarged, to protect the water supply from pollution, since it was previously found in the construction of the basin that a very large quantity of water came from the land side. Also, direct emergency connections were made with the Charles River. Incidentally, these connections have never been used.

Many leaks continued to appear in the cement mains from various causes, including rust, and, as a result, a program to relay all cement mains was begun in 1888, starting with the 16-in. leading main from the reservoir, which was replaced with 2,700 ft. of 24-in. cast-iron. This was continued each year, but not *too* rapidly.

In 1891 the right to take water from the Charles River Valley was increased to 3 mgd. Also, the construction of sewers was started. In that year 41 leaks were repaired. Only four of these were in cast-iron mains; the other 37 were in cement mains, 30 of which were

directly caused by the sewer construction. Sixteen of the leaks were the result of pick holes, and that is one reason why the writer prefers nothing but cast-iron pipe.

In 1892 a new 20-in. cast-iron force main was constructed from the pumping station to the reservoir. Also, a new 2-mil. gal. covered reservoir at a higher elevation was recommended. This, however, had to be postponed until the cement mains were all relaid.

In 1893 the well was deepened, a filter gallery was built around it, and both were covered with brick arches. Also, the pump suctions were extended into the well.

In 1894 a new water-works shop was built and we are still operating from that same shop after almost 60 years, although many changes have been made to the interior. It has also been modernized many times, and we now have a large recreation room with shower baths and lockers, which the men enjoy greatly.

By 1895 the sewerage system was complete and the daily average consumption had increased about 60% in four years, while the population increased only about 5%.

The following year trouble began to develop in service pipes under car tracks, due to electrolysis. Previous to this the city had horse cars. On one street alone 29 services were relaid and the street-car company, when shown the cause, agreed to pay two-thirds of the damage and rebond all rails to prevent further damage.

At the turn of the century, major improvements were begun at the pumping station. A new 5-mil. gal. Barr pump was purchased, and the station was enlarged to house the new pump and a new 2-mil. gal. Barr pump, to replace the original #1 Worthington pump, which had given good service for almost 30 years.

Consumption now having increased to 2½ mgd., further study was made to increase the supply. The Engineer's report recommended: (1) driven wells to supplement the existing supply; (2) beginning the installation of meters, there being only 93 meters on 3,345 services.

The year 1906 saw many more improvements. The replacing of the cement mains had progressed to a point where it was now considered fairly safe to build the new reservoir. As a result, a new 2-mil. gal. concrete tank was built near the open reservoir. This tank

was 100 ft. in diameter and 43 ft. high, with the overflow 50 ft. higher than the open reservoir. This improved the pressure downtown and also gave water to many sections of the city which were getting little or no water at all. Also in the same year the contracts were let for a new pumping station and well.

It was not until two years later that the first test was run on this new pump, a 5-mgd. Platt pump, and it was not accepted by the city for four years after that. This pump has always been a source of trouble and had to be worried along throughout its many years of operation.

In 1910 the Board of Aldermen voted to adopt a meter system to all services and the following year free water to all city departments was abolished. The right to take water up to 5 mgd. from the Charles River Valley was finally confirmed by the Legislature.

In 1913 a meter-testing machine was purchased and a repair room built at the shop. Also a 6-in meter was installed at the Walter E. Fernald State School. The sta : had been paying a flat rate of $2,000 per year and immediately the revenue from this source alone increased to $8,000.

The following year a serious break occurred on the 20-in. force main from our Pumping Station #2. This was the third break we had had in a section of 2,400 ft. of English cast-iron pipe. The break was a split in the entire 12-ft. length. This break was more serious, because of the fact that the 6-mil. gal. open reservoir was empty at that time for cleaning purposes and the break occurred on February 21, with the temperature well below zero. There were no cross-connections with our neighboring cities at that time and, as a result, a good part of the city was soon out of water. It was not long before three connections were made with the city of Newton.

In 1915 the Water, Sewer and Engineering Departments were consolidated under one head. This, in the writer's mind, has some advantages, such as laying water and sewer services in the same trench, which requires only one opening of the street, but it soon worked out that the Water Department was carrying the Sewer Department and paying a great deal of the maintenance charges, such as shop expenses, emergency men, etc. However, lately we have tried to remedy some of these discrepancies. From his years of

experience, the writer believes that the idea is good from the taxpayer's point of view, but not good for a Water Department which is trying to be self-supporting.

In 1916 an extensive study was made as to the effect of Waltham water on various types of pipes used by plumbers in houses. In the first place, it was found that brass pipe lasted only about seven years before it disintegrated, because of the unusual amount of free carbonic acid in Waltham water. The second question was: "If Waltham water can cause the disintegration of brass pipe, is it not dangerous to the human organism?" Answer: "As to the effect of free carbonic acid on the human system, it may be stated that this is the element which is generally used to put life and vim into soft drinks at the soda fountain and into various bottled waters and drinks which are so generally used and enjoyed by everyone. It is not considered harmful in these large quantities."

During 1918 water consumption went up to 2½ mgd, but later it dropped to less than two mgd. This was due to several reasons—namely, the end of World War I, a Pitometer survey, and the fact that the city was now 92.1% metered, although a population of 32,700 was being supplied with water.

In 1922 the 2-mil. gal. concrete reservoir was lined with layers of hot asphalt, felt, 6-ounce duck and a 4-in. core of brick. One of the Past-Presidents of this Association, George C. Brehm, former Director of Public Works in Waltham, gave a complete report of this work in a paper* read before the Association. When the writer emptied the reservoir 18 years later and examined the brick lining very carefully, it was found that every bit of the wall was in very good condition and that no repair work was necessary. Also, to this day there has never been any sign of a leak through the wall of the reservoir. The author highly recommends this process to stop leaks in any concrete reservoir. The only trouble during the process of lining the reservoir, due to increased pressure on the main, was a break about 2:00 a. m. on a 16-in. main in Central Square. This did cause some damage, but was soon repaired and without too much inconvenience to the public.

In the early 1920's the northern section of Waltham started to

* This JOURNAL, 38:297, 1924.

develop rapidly. This began with a camp-site development around Hardy's Pond and, because of a temporary depression at that time, people started to build permanent homes and soon demanded water. This necessitated the construction of a booster station and elevated tank. The station built at that time contained a 3-in. electric-driven centrifugal pump, with a capacity of 375 gal. per min. at a 180-ft. head, as well as a two-stage, 5-in. gasoline-driven pump with a capacity of 500 gpm. From this station the water was pumped into a 150,000-gal. elevated tank.

Also during the same year the 60-in. M.D.C. steel main was laid through Waltham. This resulted in the loss to the city of a good customer, the Walter E. Fernald State School, which has been previously mentioned.

In 1926, a 24-in. gate valve was inserted under pressure near our concrete reservoir, under the direction of the late Frank Northrup, a member of this Association. After 20 years, a much needed standby was installed at Station #2. It consisted of a 2½-mil. gal. Gould pump with a directly-driven electric motor. During this year the Pitometer Co. made a distribution and pitometer survey of the entire city.

In the fall of 1929 an extensive water-main cleaning was done by the National Water Main Cleaning Co.

In January, 1930, the author started to work for the Water Department. The population of the city at that time was 40,000; there were 6,205 services, and the city had a daily average consumption of only 2.252 mgd. We were now 100% metered, against 33% in 1917, when the consumption was the same for only 31,700 population and 4,332 services. It is evident that metering is the only thing to do unless you have plenty of water to waste. At that time we had 87.05 miles of mains and 73.83 miles of services.

The following year the National Board of Fire Underwriters made a complete survey of the city. This is the last time that this has been done in Waltham and the author recommends that a new survey be made in the near future, as conditions have changed a great deal since the last survey.

Also, we built a new high-pressure system in the westerly part of the city. This was due to the development of what we called the Cedarwood section. Two 55-hp. Fairbanks-Morse units, driving 1,000-

gpm. centrifugal pumps, were installed in a booster station. These pumps are used alternatively and pump first into a 600,000-gal standpipe about 900 ft. away; then a Ross double-action altitude valve closes, allowing water to be pumped into a 300,000-gal. tank on the side of Prospect Hill, about 9,000 feet farther away. This gives a total capacity of 900,000 gal. on the high-service area. This system is now 20 years old and is far from being overtaxed.

In 1934, a 10-in. connection was made with the Lexington Water Department on Woburn St. This connection has proved advantageous to Lexington as well as Waltham in time of emergency.

During E.R.A. and W.P.A. days many smaller mains were relayed with larger sizes, one in particular being the start of a 12-in. circumferential main around the south and east sides of the city. Some day it is hoped to complete this main, which the author has set up in a six-year major-expenditure program, as proposed recently by the Mayor.

In the year 1940 M.D.C. water became available at a higher head. This changed the situation in Waltham, as water would now be available at a suitable pressure throughout the city, except in such areas as were supplied by the Cedarwood and North Waltham high-pressure systems. This was the year in which we emptied and cleaned our concrete reservoir. We also repaired and painted the roof trusses. We found the brick lining, mentioned above, in such good condition that no repair work was necessary.

Soon we were in the midst of another war and war work throughout the city was advancing in leaps and bounds. This had its effect on our water supply, just as it did in other communities. It became necessary to call on the M.D.C. during peak loads, so as not to bring our iron content in Well #1 too high. Within two years our daily average consumption jumped over 1 mil. gal. to 3.81 mgd. The situation was becoming serious and several requests were made by this department to further the studies to enter the M.D.C. Our peak consumption was fast approaching 6 mgd. and we just could not take care of the demand without help from the M.D.C.

In 1947 the meter-repair shop was completely renovated and a new five-meter test bench was installed, replacing a 35-year-old testing machine, besides other necessary tools. This was the start of

a program to repair all stopped meters, of which we had too many, due to the war years, and to go back to a program of repairing and testing all meters which have been in service for much too long a period.

At last, negotiations were started to take our entire supply from the M.D.C. and the following year a new 12-inch force main was laid in Lexington St. to North Waltham, a distance of about two miles.

On May 23, 1949, the city of Waltham started taking its entire supply from the M.D.C. through two connections and on February 26, 1951, the new M.D.C. pumping station on Lexington St. was put in operation. During the winter a new 12-in. connection was made near the Raytheon plant in East Waltham, which greatly improved our system and gave better pressure in the center of the city. Our peak consumption has now reached as high as 7.218 mgd.

Waltham now has a population of almost 50,000 inhabitants, 114.22 miles of mains, 87.66 miles of services, about 900 hydrants, and a daily average consumption of more than 4.2 mgd.

There is still much work to be done in improving the Waltham water system, particularly in North Waltham, where there is so much new development. A new, larger standpipe is necessary, which we hope to build this year. The M.D.C. must supply more water to this section, either through Lexington or by means of larger pumps in their station in Waltham, because by the end of 1953 there will be over 800 new homes in this section of the city, all having been built in the last three years.

WASHING SLOW-SAND FILTERS WITH A LATERAL FLOW OF WATER

BY ALEXANDER J. MINKUS*

[*Read February 19, 1953.*]

The slow-sand filter was in use only a short time before it was out-numbered by the more adaptable rapid-sand filter. Probably one of the main reasons for its lack of popularity is the limited range in the character of the applied water which it can handle. Conversely, given a relatively clear water with some measure of uniformity in the turbidity and bacterial load, the slow-sand filter has a strong appeal, arising from its low operating cost and simplicity of operation. There are many instances where a large storage reservoir irons out wide fluctuations in the nature of the raw water and a small amount of additional treatment produces an excellent water.

At Hartford these conditions exist and slow-sand filtration has been the mode of water purification since 1922. Recently a method of washing the filters has been developed which reduced the sand-washing costs 22% and outage due to sand-washing operations 18%, further enhancing the desirability of slow-sand filtration. It is the purpose of this paper to describe the method.

The filtration plant of the Hartford Water Bureau now consists of 14 filter beds, plus 4 under construction. Twelve of these filters are of 1/2 acre in extent and two are 3/4 acre each. Average sand depth is 27 in., making a total of approximately 37,000 tons of sand distributed over 7 1/2 acres. The average daily draft is in the vicinity of 35 mgd and each filter requires washing about every 9 or 10 weeks.

The sand-washing procedure which was used prior to the method to be described was reviewed in the N.E.W.W.A. Journal for September, 1948.‡ In brief, it consisted of three steps: first, a raking of the sand surface with a harrow, then a washing of the top layer with a mechanical washer, and finally a washing of all the sand down

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‡ Page 242.

to the gravel with a hydraulic jet system. Various numbers of filter runs were obtained between steps.

The harrowing was done when the lost head was equal to the depth of water on the filter, which was either 6 or 7 ft. To do this the inlet of a bed was closed in the morning and the depth of water on the sand was allowed to decrease gradually by filtration, until there was insufficient head for further filtering. The filter was then drained and the sand was dried overnight. On the following day the surface of the sand was harrowed with a spike-tooth harrow, pulled by a crawler-type tractor. This operation simply broke up the surface film and mixed the dirt with the clean sand underneath. The method was satisfactory in that it was possible, in conjunction with the subsequent step, to obtain 5 or 6 runs through a filter and thus produce a total of about 700 mil gal of clear water. It was not ideal in that a filter was out of service for practically two days, and there was no removal of dirt, making it necessary to supplement harrowing with an actual washing procedure.

Washing of the sand, the second step, was carried on by a machine which washed the sand in place, while there were 24 in. of water over it. This washer was, in effect, a slow-moving tractor, carrying a device for agitating the dirty sand and a suction pump for removing the dirt, dislodged from the sand grains. The water in which the machine operated carried a considerable quantity of sand in suspension and therefore the wear and tear on the machine was exceedingly high. The greatest drawback to the use of this machine was its slow rate of operation. It required 10 days, 3 shifts per 24 hr, to wash one filter of 1/2-acre area. The main advantage was the good job of sand cleaning it did. A filter had to be washed with this machine only once every 9 to 12 months, provided it was harrowed 5 or 6 times between washes as described above.

The limitations of the two steps, harrowing and washing with the machine, lay in the relatively shallow penetration of the sand. Only the top 12 in. of sand could be kept clean and permeable, while the lower 15 in. gradually became exceedingly compact and dirty. This condition was taken care of by the third step which, for differentiation, was called reconditioning. In the reconditioning process, still in use, all the sand down to the gravel is shoveled into hydraulic

ejectors for transportation to the Nichols type of sand washers. Here the dirt is removed from the sand grains by jetting a stream of dirty sand and water against a steel plate and then separating the relatively clean sand from the dirty supernatant liquid by settling the sand and skimming off the liquid.

The reconditioning process produces a very clean sand. Once washed by this means, a filter could be kept in operation at least seven years under Hartford operating conditions, provided it was maintained by harrowing and washing with the mechanical washer in the interim. The greatest dissatisfaction with the whole washing scheme was with that part of it requiring the use of the mechanical washer.

During the many times it was necessary to wade out to the washer after it had broken down in some remote corner of the filter bed, it was noted that the dirt on the sand surface was readily suspended in the water by rubber boots scuffing over the sand, and that the dirt was slow in settling out. The thought occurred that if a fast-moving mechanical device would stir up the sand, and if a lateral flow of water could be provided at the same time, then a considerable amount of dirt would be carried away to some discharge point.

It was relatively easy to provide a stirring device. A spring-tooth harrow, drawn by the same small crawler-type tractor used in the previous method, was the solution. The harrow has 16 teeth, 1 3/4 in. wide, a maximum penetration of 8 in. The tractor is driven by a gasoline motor, rated at 20 hp at the draw-bar, weighs 3,300 lb, and has a ground pressure of 620 lb per sq ft.

To provide the horizontal flow of water over the sand surface required some improvisation. In the corner of each filter there is a float-controlled altitude valve, which normally supplies the raw water. However, the drain for the raw water is also in the same corner. Thus it is impossible to provide a continuous flow of water across the sand surface without the use of a pump, there being no other inlet or outlet. Since no portable pumps of the required capacity were readily available, the idea of a continuous horizontal flow was given up for one of intermittent flow, which actually works satisfactorily.

The intermittent flow is obtained by filling the filter with raw water to a depth varying between 9 and 10 in. The influent valve is

closed and the drain valve, located in the same corner of the filter, is opened wide. The tractor operator then starts to harrow the sand along the wall farthest from the drain and moves in parallel paths progressively closer to the drain. As the 1/2-acre filters are 143 × 169 ft in plan and the diagonal distance is 221 ft, the dirt first stirred up by the tractor and harrow settles out before reaching the drain 200 ft away, but it is again stirred up as the tractor approaches the drain. Long before the tractor itself reaches the drain the dirty water is flowing out of the bed.

If harrowing operations are started with water 10 in. deep and suspended when it has reached 2 or 3 in., three traverses of the bed can be made. The second traverse is at right angles to the first and third passes. As much of the remaining 2 or 3 in. of water as the rough surface of the sand permits is drained off after the tractor stops moving. During these operations it is possible to maintain a reverse flow of 1/2 mgd in a 1/2-acre filter, equivalent to 1/40th of an inch rise per minute. That of itself has a negligible washing effect, but it is done with the thought of providing all the water possible for the horizontal flow and yet not softening the sand enough to bog down the tractor.

About 90 min are required to make the 3 traverses and after all the dirty water that can be is drained from the surface, the filter is filled again with clear water from the inlet valve to a depth of 10 in. and the operation is repeated. This time the initial direction of travel is at right angles to the last traverse and three more traverses are made before the water is too low again. The total combination of movements covers the bed 6 times, 3 in each direction.

By the time the second round of traverses is completed, the 8-hr working day is over and the filter sand is allowed to dry overnight. The next day the filter is harrowed in the dry, first with a 24-in. diameter disc harrow, and then with the spring-tooth harrow. The disc harrow loosens up the compact sand to a depth of about 10 in. and the spring-tooth harrow smooths over the sand surface. Two passes are made with each harrow. The filter is then ready for filling and by 6 P.M. of that day it is ready for operation. The two men assigned to these operations spend the remainder of their 8-hour day greasing the tractor and moving it to the site of the next job.

Since each filter is filled twice to a depth of 10 in. and there is a reverse flow of 1/40 in. rise per min over a 3-hr period, the total wash-water volume for a half-acre filter is approximately 350,000 gal. Since the filter runs average about 160 mil gal, the wash water amounts to 1/4 of one percent.

The average rate of flow of the wash water over the sand surface is about 1,300 gal per min. Because of the large cross-sectional area of the stream, the horizontal velocity is very low, probably in the range of one foot per minute, but the progressive agitation of the sand in the direction of stream flow makes it possible to keep the dirt in suspension until it reaches the drain. The highly turbid appearance of the wash water as it enters the sewers indicates that large quantities of dirt are being removed. During 1951 and 1952, the average turbidity of this water was 1,600 ppm, with a peak value of 8,000 ppm.

The simple type of equipment used in this process is illustrated in Figures 1 to 4. The spring-tooth harrow employed to agitate the sand is shown in Figure 1. The spike-tooth rake at the rear of the harrow smooths over, to some extent, the furrows created by the spring teeth.

Figure 2 shows the spring-tooth harrow and tractor in a filter bed during the operation. The dark color of the water illustrates the large amount of dirt suspended in the water.

Figure 3 is a view of the disc harrow which serves to soften the washed sand and Figure 4 shows the disc harrow in operation.

To check the effectiveness of the lateral-flow washing procedure, a vertical core of sand is taken from the top 12 in. in each filter immediately after washing. The permeability of this core is tested in the laboratory in an apparatus patterned after one described by Professor Glennon Gilboy.* The numerical value of the permeability is measured in feet per day and is equivalent to the Darcy coefficient for seepage through soils for a unit area and unit slope.

The laboratory tests show that the horizontal-flow washing method does not keep the sand at a uniform degree of permeability, but the rate of change in the permeability is retarded enough to make the process economically attractive. To illustrate this point, Figure

* Proceedings American Society of Civil Engineers, Oct., 1931.

FIG. 1.—SPRING-TOOTH HARROW, EMPLOYED TO AGITATE SAND IN FILTER BED.

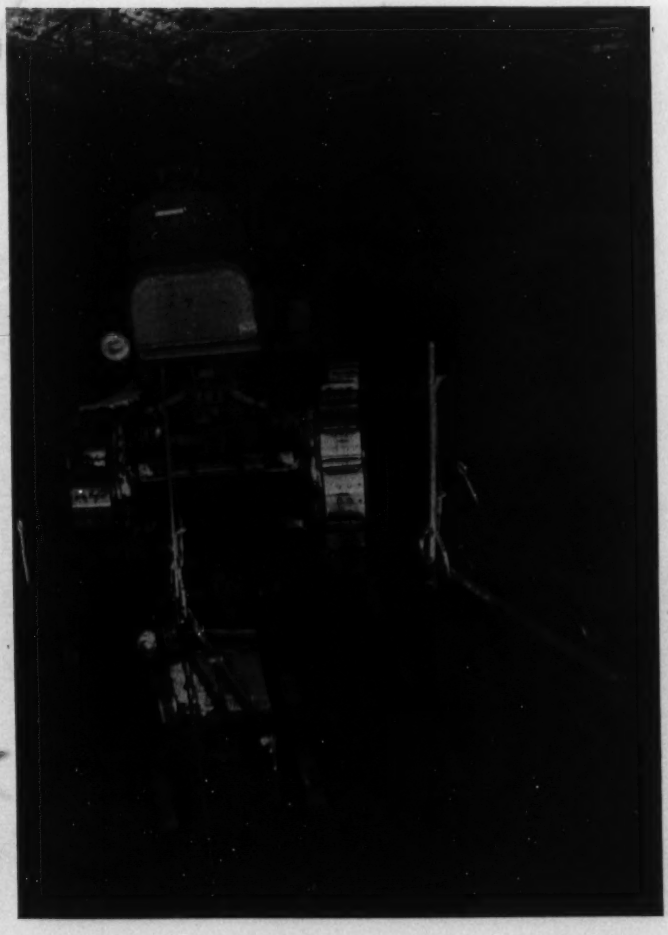

FIG. 2.—SPRING-TOOTH HARROW AND TRACTOR IN FILTER BED.

FIG. 3.—DISC HARROW, USED TO SOFTEN WASHED SAND.

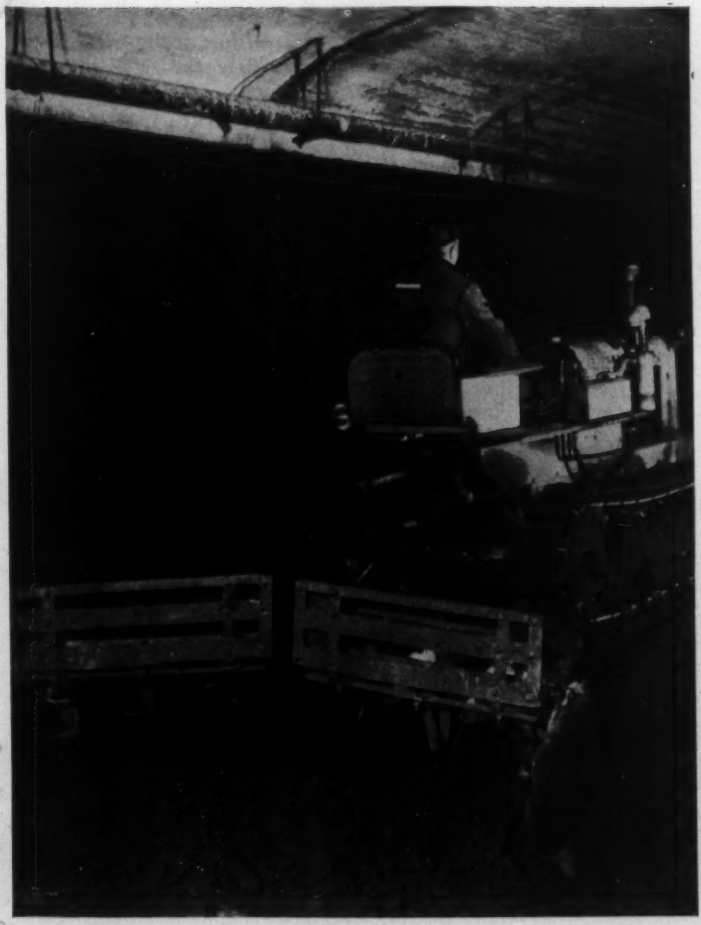

FIG. 4.—DISC HARROW IN OPERATION.

5 shows the permeability plotted against the volume of water filtered through two filters. The filters were cleaned identically with the so-called reconditioning process at approximately the same time and then put into service to filter the same type of water. Filter No. 11 was maintained with the horizontal-flow method of washing and its record is shown in the upper curve. Filter No. 12 was simply harrowed in the dry every time the lost head reached 7 ft. Its record is shown by the lower curve.

As was to be expected, the permeability of the sand in Filter No. 12 decreased rapidly, because the dry harrowing was not removing any dirt, and by the time it had filtered 1,800 mil gal, the filter was completely plugged up. In contrast, Filter No. 11, maintained with lateral-flow washing, has delivered 3,400 mil gal to date and is still in satisfactory condition.

The effectiveness of horizontal-flow washing is further illustrated in the record of Filter No. 12. After it had become plugged, it was

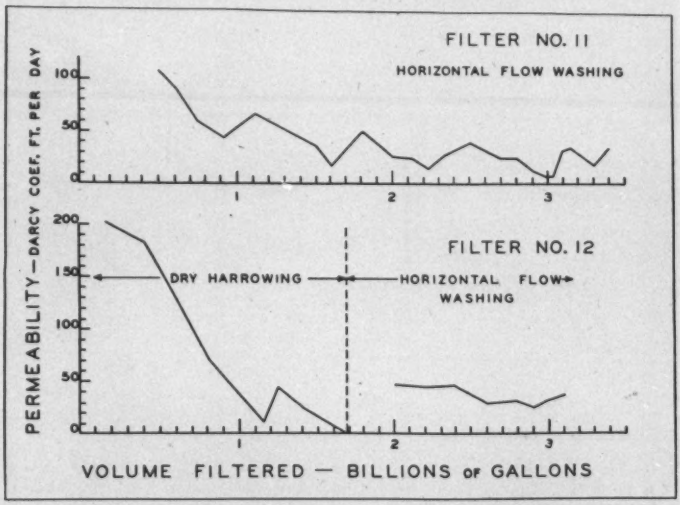

FIG. 5.—PERMEABILITY PLOTTED AGAINST VOLUME OF WATER FILTERED.

washed once with a horizontal flow, placed into service, and maintained thereafter with horizontal-flow washing. It has filtered 1,300 mil gal since the first horizontal-flow wash and is still going strong.

To make a thorough evaluation of this method of sand washing, it is necesary to consider its effect on the quality of the filter effluent. For this purpose Figure 6 is included, showing the annual averages of color, total bacteria at 20°C., coliform content, and rates of filtration per acre of sand during the past 30 years. All these data are for the effluent of the filters, taken prior to the addition of chlorine, so that its sterilizing properties would not mask the action in the filters.

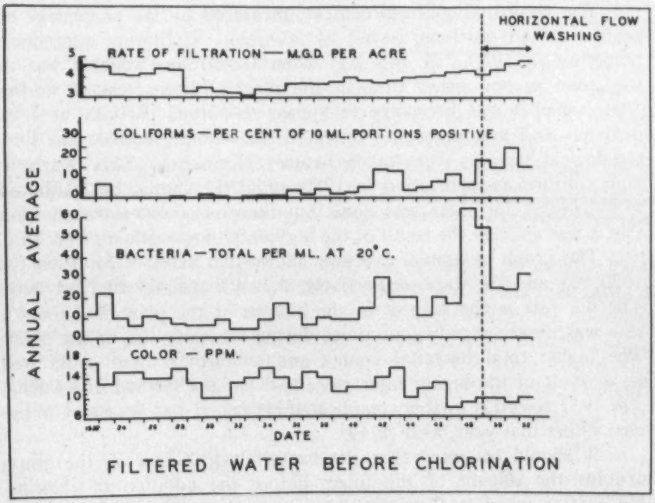
HORIZONTAL FLOW WASHING
RATE OF FILTRATION — M.G.D. PER ACRE
COLIFORMS — PER CENT OF 10 ML. PORTIONS POSITIVE
BACTERIA — TOTAL PER ML. AT 20°C.
COLOR — PPM.
ANNUAL AVERAGE
DATE
FILTERED WATER BEFORE CHLORINATION

FIG. 6.—EFFECT OF HORIZONTAL-FLOW WASHING ON QUALITY OF FILTER EFFLUENT.

It is readily apparent that there was no material change in the color of the filter effluent resulting from the use of the lateral-flow washing method. The average color of 10 for the past three years

is the same as that of the previous 3 years within the limits of accuracy of this test.

The total bacterial count on agar at 20°C. averaged 26 per ml for 1950, '51 and '52 and is slightly higher than the average prior to horizontal-flow washing. The exceptionally high bacterial count in 1948 was caused by the experimental chlorination of the raw water applied to one of the filters.

The pre-chlorination caused a large amount of sloughing from the sand grains, accompanied by high bacterial counts, until a chlorine residual began to appear in the effluent of this filter. By that time the large numbers of bacteria added to the plant effluent from this one filter had materially changed the average for the entire plant.

The record of coliform content, measured by the percentage of positive 10-ml portions, is not so favorable. Coliforms were more numerous in 1950, '51 and '52, when lateral-flow washing was in use, than at any other time in the plant's history, except during 1949, when it was necessary to bypass the filters partially at peak demands and add untreated water to the storage basins. The bacteriological samples were taken before chlorination. This relatively high coliform concentration in 1950 and 1951 cannot be attributed to bypassing, for there was none, but there is considerable question that it was entirely the result of the horizontal-flow method of washing.

The graph of annual averages shows that rates of filtration for 1950, '51 and '52 were, respectively, 4.3, 4.7 and 4.8 mgd per acre. The 4.8 rate is the highest in the history of the plant and the 4.3 rate was exceeded only four times during the early life of the plant. The higher total bacterial counts and coliform content could well be a result of the higher filter rates and not the method of washing. The 1952 record is further complicated because it was necessary to bypass filters that year, as in 1949.

It should be noted that the bacteriological data on the graph are for the effluent of the filters before the addition of chlorine. No matter whether the bacterial increases are due to lateral-flow washing or high filter rates, the extent of the increase was small and required no perceptible increase in chlorine dosage to produce an effluent well within U. S. Public Health standards at all times. Prior to 1941, when there was no chlorination, an increase in the coliform index would have been significant, but with the present policy of em-

ploying chlorine as an additional safeguard to slow-sand filtration, the increase in bacterial content of the magnitude experienced is of little consequence.

In addition to keeping the sand suitably clean, the horizontal-flow method has reduced the outage of the beds considerably. A bed is now out of service 36 hr for washing purposes. That is a considerable improvement over the old scheme, which required 10 days' outage for washing one filter, plus 5 days per year to harrow the sand in the dry between washes. The total annual outage for the entire plant, including the time required for the reconditioning process, which is common to any method of washing, averaged 479 bed-days for the two years 1947 and 1948, when the old scheme of washing was in use. The corresponding figure for two years of horizontal-flow washing, 1950 and 1951, was 395 bed-days, or a reduction of 18%.

The reduction in lost time is due both to a speeding up of the washing process and to an increase in the length of runs. Table 1 shows that 119 mil gal were obtained annually per bed per treatment with the old scheme, compared with 161 mil gal following horizontal-flow washing. The word "treatment" is used to indicate any entry into a filter bed, whether harrowing, mechanical washing, reconditioning, or horizontal-flow washing. The increase in water delivered per treatment is, therefore, 35%.

The increase in filter runs and the reduction in outage have reduced sand-washing costs. Table 2 shows that the overall cost of washing sand with the lateral-flow method, including the so-called reconditioning process, was $2.01 per mil gal of water filtered. Comparable costs for the old scheme, computed at the same wage scale, were $2.58 per mil gal, or a difference of 22%. With a total volume filtered annually of about 13 billion gallons, this has effected an an-

TABLE 1.—FILTER RUNS PER TREATMENT

Year	Old scheme 1947	1948	Horizontal-flow 1950	1951
Volume Filtered (m.g.)	11,425	12,659	11,706	12,986
No. of Treatments	109	95	74	79
Volume per Treatment (m.g.)	105	133	158	164
Average Volume per Treatment (m.g.)	119		161	

TABLE 2.—WASHING COSTS PER MILLION GALLONS OF WATER FILTERED

	Old scheme			Horizontal-flow		
	Harrowing and mechanical washing	Reconditioning	Total	Cross-flow	Reconditioning	Total
Labor	$0.68	$1.44	$2.12	$0.30	$1.44	$1.74
Gas, oil, electricity	0.02	0.09	0.11	0.01	0.09	0.10
Repairs	0.24	0.11	0.35	0.06	0.11	0.17
	$0.94	$1.64	$2.58	$0.37	$1.64	$2.01

nual saving of $7,400 in washing costs. Total purification costs in 1952, including this sand washing, operation and maintenance of plant, and chlorination, amounted to $10.90 per mil gal.

Attention is directed to the fact that the washing process, as described above, has been carried on by improved means in a plant with no specific provision for its use. The results have been so encouraging that the design of four new filters, now being constructed at a cost of $1,200,000, incorporate a means of establishing a continuous lateral flow over the sand surface. This will be done by two parallel channels, located along opposite walls in each filter. One channel, 219 ft long, will be the inlet, discharging water from one side of the channel through submerged openings, to prevent displacement of the sand. The other channel, of the same length, will carry away the dirty water to a sewer. The rim of this channel can be adjusted in height, thus constituting a control over the depth of water on the sand.

As these new filters have not been completed at this time, the full value of lateral-flow washing remains to be determined. At any rate there has been, to date, a most worth-while reduction of 18% in outage and a cost reduction of 22% in the older portion of the plant, and it is expected that the new filters will do considerably better.

William A. D. Wurts is the Manager of The Metropolitan District of Hartford, and Warren A. Gentner is the Deputy Manager and Chief Engineer of the Water Bureau. These improvements in procedure and design have been made under their direction and with their encouragement.

IMPROVEMENTS TO PUMPING AND FILTRATION FACILITIES AT CAMBRIDGE FILTER PLANT

BY THOMAS R. CAMP*

[*Read March 19, 1953.*]

At the Annual Convention of the New England Water Works Association in 1951, a paper was read by Howard M. Turner, President of the Cambridge Water Board, describing all the recent improvements to the Cambridge Water System, including the new works at the Cambridge Filtration Plant. Because of the broad scope of Mr. Turner's paper, it was not possible for him to treat any of the new works in great detail. For a general description of all the new works the reader is referred to Mr. Turner's paper.‡ It is the purpose of the present paper to describe in more detail the new pretreatment works and the new high-lift pumping station, and to present some of the reasons why these improvements were finally constructed as they were.

The old pumping station shown on the General Layout, Figure 1, was a steam plant, which at the time of its abandonment contained as a service unit a Worthington, horizontal, cross-compound steam pump of 20-mgd capacity, which was installed in 1928, and a standby, triple-expansion, Leavitt vertical pump of 20-mgd capacity, installed in 1896. This plant originally took suction directly from Fresh Pond. Since the completion of the filter plant in 1922, the old pumping station has taken suction through a 48-in. pipe from the filtered-water reservoir at a point near the new transformer substation, as shown on Figure 1.

In 1946 the Cambridge Water Board authorized Camp, Dresser & McKee to make alternate studies for the improvement of its high-lift pumping facilities, leading to more efficient and less costly operation and maintenance. When these studies were started, it was hoped to modernize the old pumping station by means of centrifugal pumping equipment. No consideration was given to a new pumping sta-

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‡ This JOURNAL, 66:2, 142, June, 1952.

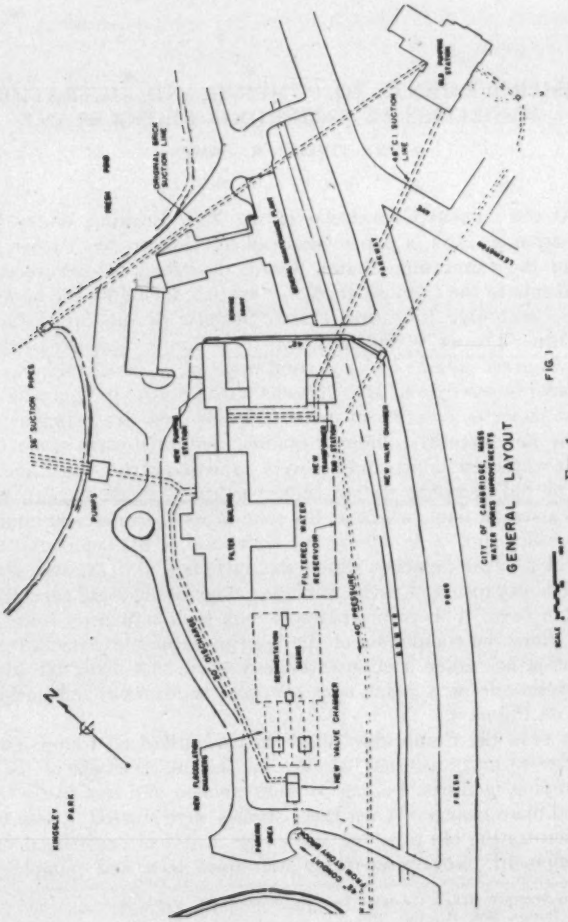

CITY OF CAMBRIDGE, MASS
WATER WORKS IMPROVEMENTS
GENERAL LAYOUT
FIG. I
FRESH POND
ORIGINAL BRICK SUCTION LINE
36" SUCTION PIPES
PUMPS
FILTER BUILDING
FILTERED WATER RESERVOIR
NEW VALVE CHAMBER
NEW FLOCCULATION CHAMBERS
PARKING AREA
KINGSLEY PARK
FRESH POND
PARKWAY
LEXINGTON AVE
48" C I SUCTION LINE
SCALE

FIG. 1.

tion at the filter plant at that time, because the Water Board had under consideration the construction of a new sedimentation basin on the land now occupied by the new pumping station. Plans for this sedimentation basin were prepared in 1935, but no decision had been made for its construction.

Four alternate schemes were considered in our 1946 studies for improving the high-lift pumping facilities, which included: (1) electric motor-driven, standby centrifugal pumps in the existing station, (2) steam turbine-driven, standby centrifugal pumps in the existing station, (3) a new station with electric motor-driven centrifugal pumps, and (4) a new station with Diesel engine-driven certifugal pumps. In each of the two alternates for standby centrifugal pumps in the existing building, two 12-mgd pumps were proposed, while in each of the two alternates for the new station, three 12-mgd pumping units were proposed.

The 1946 studies indicated that the total annual fixed charges and operating costs for remodeling the old station would be greater than the annual cost of operating the existing station, whereas the total annual fixed charges and operating cost of a new electric station or a new Diesel station would be less than the operating cost of the existing station. It was proposed to locate the new station, either Diesel or electric, on the old pumping-station land, just southwest of the old pumping-station building. Our studies showed that the first cost of a new Diesel station would be considerably greater than the first cost of a new electric station, but the total annual cost of a new Diesel station would be less than the total annual cost of a new electric station. The results of our studies were presented in a formal report, dated May 27, 1946. No decision was made that year.

In 1947 we were authorized by the Cambridge Water Board to restudy the matter of flocculation and sedimentation at the filter plant and to report upon the feasibility of constructing a new high-lift pumping station on the filtration-plant grounds. The results of this investigation were presented in a formal report, dated October 9, 1947.

It should be noted in this connection that the 1935 plan for increased sedimentation capacity contemplated a new sedimentation basin of about the same depth, but occupying about twice the area of

the existing sedimentation basins. There was no place to construct such a basin on the filter-plant grounds, except at the site now occupied by the new pumping station. A study of the general layout on Figure 1 will indicate that if such a basin had been constructed, it would have occupied so much land that it would have been difficult to locate a new high-lift pumping station on the filtration-plant land and impossible to construct the new general maintenance plant on the filtration-plant land.

At the time the filtration plant was constructed in 1922, 10 filters were provided, with a total nominal capacity of 14 mgd at a filtration rate of 2 gpm per sq ft. The sedimentation basins, with a capacity of 1.5 mg, provided a detention period of 2.5 hours at the plant capacity of 14 mgd. At the time of the original construction the need for gentle mixing over a considerable period of time for floc formation was not understood by the profession. Consequently the alum solution was added into the influent pipe to the sedimentation basins and no provisions for mixing were made.

In 1932 it became necessary to increase the capacity of the filter plant by the addition of six new filters. The nominal capacity at the 2-gpm filtration rate was thus increased to 22.4 mgd and the detention period in the sedimentation basins was correspondingly reduced to 94 minutes. The corresponding overflow rate was approximately 1,850 gpd per sq ft of sedimentation-basin floor area.

The performance of the basin was poor and resulted in short filter runs. Studies made under the author's direction in the spring of 1932 by Lawrence F. Wagner, a student at M.I.T., showed that the basin was removing about 32% of suspended solids and that about two-thirds the detention time was being used for completion of flocculation. The Water Board's plans of 1935 included the construction of so-called mixing or flocculation basins just ahead of the old sedimentation basins, as well as the new sedimentation basin north of the filter plant.

In 1904 the late Allen Hazen showed that the removal of discrete suspended particles in a sedimentation basin was a function of the overflow rate and not the detention period (Transactions A.S.C.E., December, 1904, p. 63). This means that removal is a function of floor area and independent of the depth. Hazen, himself, did not fol-

low his studies through, to determine whether this conclusion would apply to removal of flocculant particles, and the practice still prevails of fixing the dimensions of settling tanks on the basis of standard detention periods. The author has been studying sedimentation both experimentally and theoretically for about 20 years. In 1946 he presented evidence to show that, even with flocculant suspensions, removal is a function of floor area and nearly independent of the depth, provided the depth is not made so small as to promote scour of the sludge (Transactions A.S.C.E., 1946, p. 895).

It is obvious from the foregoing discussion that the proper solution of the sedimentation problem for the Cambridge Water Works was to increase the sedimentation-basin floor area by inserting intermediate floors (or trays) in the existing sedimentation basins. The adoption of this plan of procedure released the entire area north of the present filter building for the construction of the new high-lift pumping station and the general maintenance plant. The land occupied by the old pumping station is now available for other uses; and the yard and buildings on New Street, which were formerly used for general maintenance purposes, are also available for other use. The old pumping station and site have just been sold for approximately $60,000.

Figure 2 shows the alterations to the sedimentation basins as finally constructed. The two intermediate trays served to reduce the overflow rate from about 1,850 to 690 gal per day per sq ft of settling-tank floor area at the nominal plant capacity of 22.4 mgd. Sludge is removed from the basins at intervals of about two weeks by unwatering a basin and flushing by means of monitor nozzles, mounted on the floors. This is a hand operation and requires the operating personnel to enter the sedimentation basins. The headroom between the floor and ceiling of each pass is none too great, particularly on the top deck. Consideration was given in design to the construction of a new roof at a higher elevation, in order to provide more headroom, but the cost proved prohibitive.

Flocculated raw water enters the bottom pass of each sedimentation basin from two of the new flocculation chambers through two rectangular sluice gates. With both sedimentation basins in operation at the nominal plant capacity of 22.4 mgd, the mean velocity

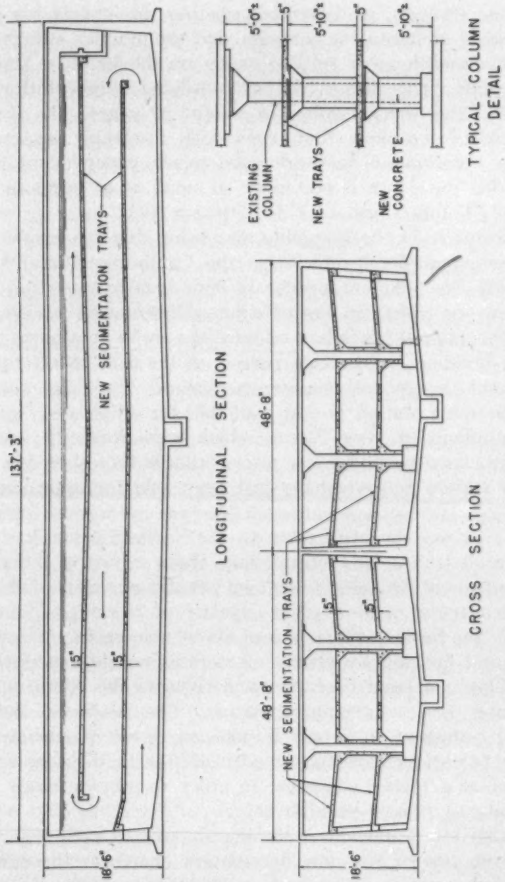


FIG. 2.—ALTERATIONS TO SEDIMENTATION BASINS.

in the two lower passes of the sedimentation basins will be about 4.1 ft per min and the mean velocity in the upper pass will be about 5.8 fpm. If one basin is out of operation for cleaning, the mean velocity in the upper pass of the other basin will be increased to about 11.6 fpm and the overflow rate will be increased to about 1,380 gpm per sq ft. Under such conditions sludge starts to scour, the quality of the effluent deteriorates, and a slight decrease in the lengths of filter runs results, as shown by plant operating records.

Figure 3 shows the operating layout of the sedimentation basins

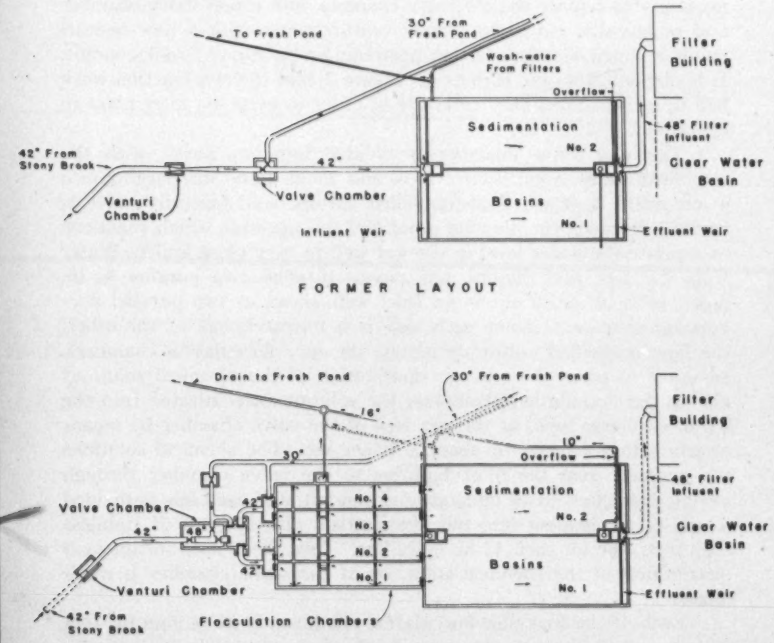


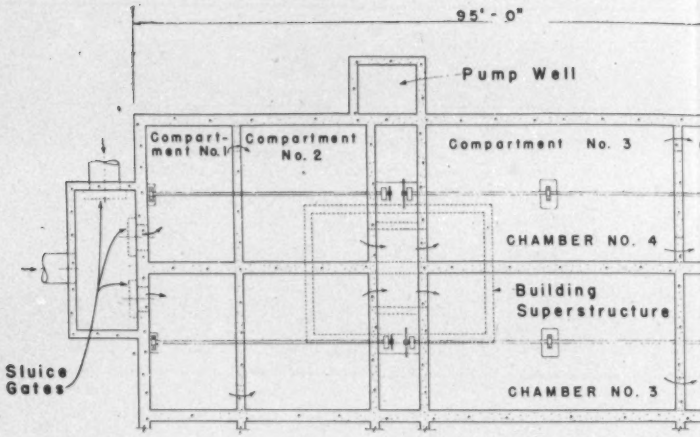
FIG. 3.—OPERATING LAYOUT OF SEDIMENTATION BASINS.

prior to the recent construction program, and with the new flocculation chambers completed. It will be noted in the upper plan that raw water is taken by gravity from Stony Brook and is also pumped through a 30-in. force main from Fresh Pond. Prior to the new construction program, these two conduits came together in a valve chamber, at which a solution of alum was added, and the mixture was then conveyed through a 42-in. pipeline into the influent flume of the sedimentation basins.

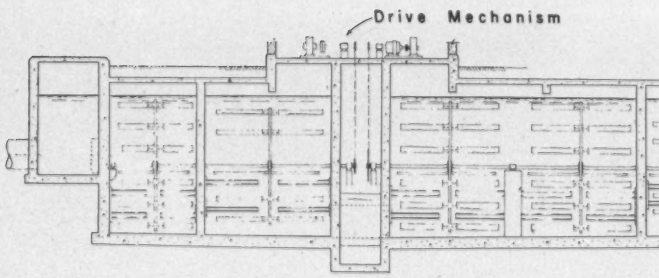
The new flocculation chambers occupy so much room that it was necessary to replace the old valve chamber with a new valve chamber and replace the old Stony Brook venturi meter with a new venturi meter at a new location, farther upstream on the Stony Brook conduit. It is obvious from the layouts on Figure 3 that the construction work had to be scheduled very carefully in order to keep the filter plant in operation.

The new valve chamber is divided into two parts, with the combined water from Stony Brook and Fresh Pond discharging into a wet well. A 48-in., float-controlled, air-operated butterfly valve is installed between the junction point and the wet well, which functions to regulate the water level in the wet well to very close limits. Water from the wet well divides and passes through two parallel 42-in. pipes, each of which enters an inlet well ahead of two parallel flocculation chambers. Since each side is a mirror image of the other, the flow is divided uniformly among the four flocculation chambers. In order to effect the uniform distribution of the chemical solutions among the flocculation chambers, the solutions are injected into the 42-in. discharge pipes at the wet well of the valve chamber by means of jets with a velocity of about 8 ft per sec. The chemical solutions are pumped from the filter building to the valve chamber through flexible hose lines in an underground conduit, and each line is divided at the valve chamber into two symmetrical pipe systems of stainless steel pipe, one for each 42-in. main line. Thus the chance for unequal distribution of the chemical solutions at the valve chamber is minimized.

Each of the four new flocculation chambers is 95 ft long by 16.5 ft wide and has an average water depth of about 16.6 ft, as shown in Figure 4. Each chamber is divided into four compartments, having

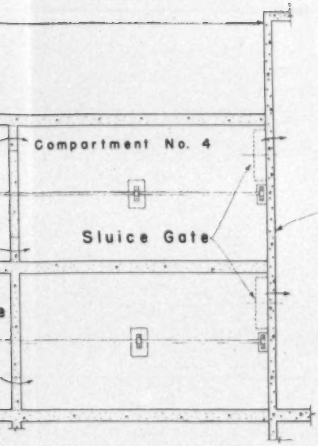


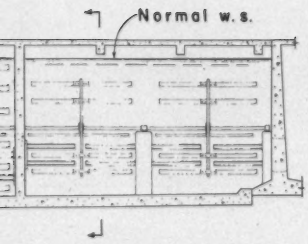
HALF PLAN

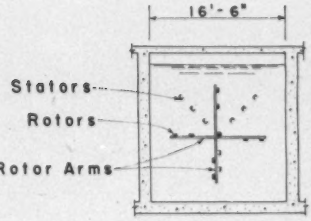


LONGITUDINAL SECTION

FIG. 4.—NEW FLOCCULA







CROSS SECTION

ON

CULATION CHAMBERS.

full speed with the water still, the power requirements under starting conditions would be 8 times the power loss obtaining after the water comes to equilibrium at half the velocity of the rotors. In order to avoid this initially high power consumption, which would be of no value in flocculation, variable-speed motors were provided, with automatic controls under starting conditions, to prevent bringing the motors up to full speed in less than five minutes.

Since nothing was known about the relative velocity of the rotors and the surrounding water and of the amount of power loss to be expected in the transmission bearings, and stuffing boxes, provisions were made in the specifications for a test to evaluate the unknowns, after the first chamber was equipped with the flocculating mechanisms, and for changing the size of sprocket wheels.

It will be noted from Figure 4 that there are two shafts for driving the flocculating rotors in each chamber, a high-speed shaft for the smaller compartments and a low-speed shaft for the larger compartments. With the sprockets finally selected, the speed of the high-speed shaft is 78% greater than the speed of the low-speed shaft. The mechanism for each chamber is driven by a single, 10-hp, variable-speed electric motor of the movable-brush type, having a 3-to-1 speed range. The minimum speed of the motor is approximately 833 rpm and, after testing, the stop on the speed change was set to limit the top speed at 2,200 rpm.

The tests on the flocculator mechanism showed that approximately 1 hp is lost through transmission, stuffing boxes and bearings, regardless of speed, and that after equilibrium is established, the water revolves at about 24% of the speed of the rotors in the high-speed compartments and at about 32% of the speed of the rotors in the low-speed compartments. There is insufficient operating experience so far to show what speeds are best for floc formation, but the indications are that the lower part of the available speed range will be preferable.

The mean velocity gradient, used herein as a measure of the adequacy of flocculation, has the dimensions of feet per second per foot. It was proposed in the design to provide flocculation mechanisms which would produce velocity gradients with a 2-to-1 range from maximum to minimum, with change in speed of motor and with

maximum values of 120 in the first compartment, 80 in the second compartment, 40 in the third compartment, and 20 in the fourth compartment. Thus the water entering the sedimentation basins would have been subjected to a velocity gradient in the fourth compartment of somewhere between 10 and 20 ft per second per ft. These values seemed to be indicated as proper for design by our study of the power provisions for flocculation basins at other plants throughout the country. The minimum velocity gradient obtainable in the fourth compartment with the equipment as finally adjusted will be about 5 in the winter and about 7.5 in the summer.

The final results show that the velocity gradient in the first compartment will be 5.63 times the velocity gradient in compartment four, instead of 6 as originally proposed. The velocity gradient in the second compartment will be 3.81 times the velocity gradient in the fourth compartment, instead of 4 times this velocity gradient as originally proposed, and the velocity gradient in the third compartment will be 1.92 instead of 2 times the velocity gradient in the fourth compartment.

A comparison of the plant operating results for the period from 1941 to 1950, inclusive, before the recent improvements, with the operating results for the period from April to December, 1952, inclusive, shows that filter runs have been increased from an average of 15.25 hr to 31.48 hr, or more than doubled. The wash water has been cut from 5.14% to 2.39% of the amount filtered and the alum dose has been reduced about 10%. The value of the saving in wash water and alum, based on the cost of Metropolitan water at $40 per mg, is about $10,000 per year.

The new high-lift pumping station is shown in plan and section on Figure 5. Prior to the construction of this station, lime for corrosion abatement was added to the aerator effluent at the west corner of the filter building, at which point chlorine for disinfection is also added. This is a proper point for post-chlorination, in that the detention period in the filtered-water reservoir is available for bacterial kill before the water is pumped into the distribution system. The high pH value, resulting from lime treatment, however, made the chlorine relatively less effective than it should be. Hence, when the pumping station was constructed, the point of application of lime was

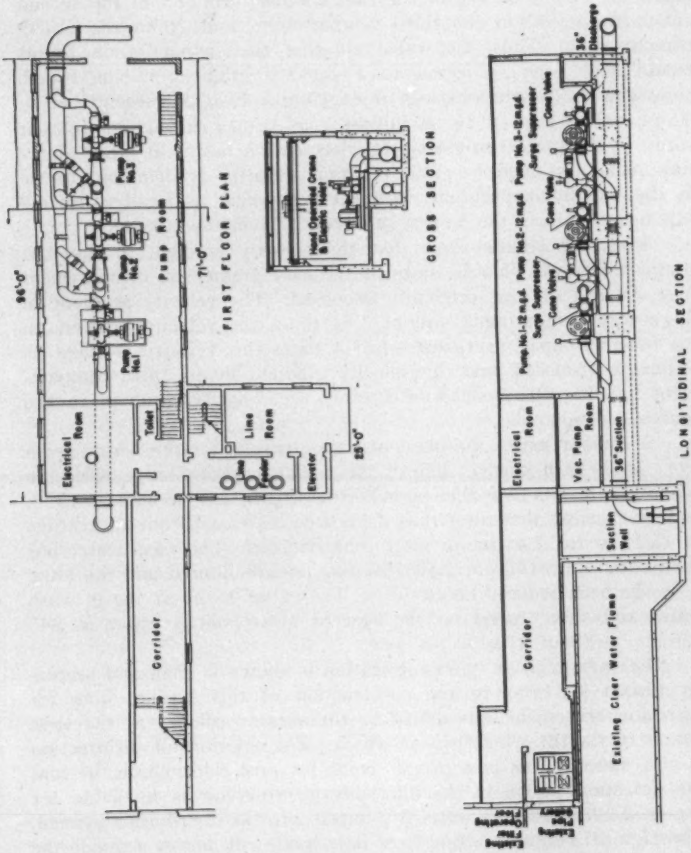


FIG. 5.—HIGH-LIFT PUMPING STATION.

moved to the pump suction well and lime-feeding facilities and storage space were provided in the new building, as shown in Figure 5.

The pumping equipment consists of three, single-stage, horizontal centrifugal pumps, direct-connected to 500-hp synchronous motors, each with a capacity of about 12 mgd against a total pumping head of approximately 180 ft. Premium efficiency motors were specified and the wire-to-water efficiency throughout the operating range is 83% to 86%. The pump operation is controlled automatically by means of float switches on the suction well, so that one pump will remain in operation continuously and a second pump will operate intermittently, to maintain the water level in the clear-water reservoir within a range of approximately 1 ft. The third pump is a standby. All three pumps may operate together, but the need for the capacity of all three pumps together will be rare.

Each pump is provided with a separate dry-feed machine for feeding lime into the suction well which starts and stops automatically with the pump motor. The lime building has two floors, with storage space on both floors and the feeder hoppers on the upper floor.

The main piping system in the pump room consists of welded steel pipe, with cement lining on the interior and with Dresser couplings for flexibility. Hydraulic-operated cone valves are used as check valves for the pumps, and they are controlled automatically to close slowly just before a pump motor stops and to open slowly just after a pump motor starts, with an opening and closing time of approximately 30 sec. In case of power failure the cone valves close automatically in a period of about 3 sec.

In view of the fact that the Cambridge distribution system is a low-pressure system, it was considered necessary to limit water-hammer pressures so that the maximum pressures would not exceed those to which the system had so far been subjected. For this purpose, two Pelton surge suppressors were installed, as shown in Figure 5, to take the backflow in case of power failure and to limit the pressure in the discharge piping to not more than 87 psi. The surge suppressors have met the specifications and the need for them has been demonstrated already.

Electric-powered pumps were selected instead of Diesel-powered pumps in the final design because of the relative freedom from noise

and fumes. It will be noted that no standby source of power has been provided. The reason for this is that electric power is available from either of two underground power conduits of the Cambridge Electric Co. The probability of a power outage on both transmission lines at the same time is remote. Moreover, an interruption in operation of the pumping station for a short period will not deprive the city of water, because of the storage capacity of the Payson Park Reservoir and the availability of Metropolitan water through three connections to the Metropolitan system.

The general contractor for the construction of both the improvements to the sedimentation facilities and the pumping station was the R. Zoppo Co. The pumps, motors and station switchboard were furnished by the Turbine Equipment Co. of New England. The flocculation equipment was furnished by the Link Belt Co. The final cost of construction of the flocculation basins and improvements to the sedimentation basins was $486,461 and the cost of the flocculation equipment was an additional $45,415. The cost of construction of the high-lift pumping station was $321,896 and the additional cost for the pumps, motors and switchboard was $45,903. The author's partner, Darrell A. Root, was resident engineer during the construction program.

THE NEW BRITAIN WATER FILTRATION PLANT

BY I. LAIRD NEWELL*

[*Read April 16, 1953.*]

The Water Commissioners of the city of New Britain, Conn., are required to serve a population of about 80,000. This is a highly industrialized community and more than 40% of the water consumed is used by the factories. The consumption has steadily increased over the years to the present rate of about 9.3 mgd.

Three reservoirs and three well systems provide the necessary water for the city. Wolcott Reservoir and Patton Brook Well are connected by a pipeline to Shuttlemeadow Reservoir, which has a storage capacity of 1,290 mil gal. Water from Whigville Reservoir and the White Bridge Well supplies is conducted by a pipeline to Shuttlemeadow Reservoir. However, if desired, these supplies can also be connected directly to the filter plant or to the distribution system. Typical sanitary analyses of these supplies are given in Table 1. The raw water, which may be a mixture of both reservoir

TABLE 1.—TYPICAL SANITARY ANALYSES OF WATER FROM NEW BRITAIN SOURCES

	Whigville (unfiltered but chlorinated)	Shuttlemeadow (filtered)	White Bridge (well supply)
Color	12	3	3
Odor	None	None	None
Turbidity	1	1	1
Nitrite Nitrogen	0.001	0.002	0.002
Nitrate Nitrogen	0.10	0.07	0.4
Ammonia Nitrogen	0.008	0.016	0.008
Albuminoid Nitrogen	0.32	0.032	0.024
Chloride	3.2	2.8	2.8
Alkalinity	9.0	16	25
Hardness	14	36	38
Iron	0.3	0.2	less than 0.1

* Research Chemist, Henry Souther Engineering Co., 11 Laurel St., Hartford 6, Conn.

and well supplies, is delivered to the filter plant by a Diesel-powered pumping station, located below the dam at Shuttlemeadow Reservoir.

The sanitary quality of the waters from these supplies has always been good, and copper-sulfate treatment of the reservoirs and chlorination of both reservoir and well waters has been practiced. However, there have been the usual complaints of odors, taste and color which are frequently associated with surface-water supplies.

In late 1938 the Board of Water Commissioners engaged the services of The Henry Souther Engineering Co. as consulting engineers in the design of a filter plant for New Britain. After a study of the needs of the city, the character of the water available, and the types of filters available, it was decided to build a 10-mgd, Morse Type, rapid-sand filter plant of steel construction. Influencing this choice was the fact that a Morse Type plant of steel construction had been completed and put into operation at Bristol, Conn. The excellent results from the operation of the Bristol plant, as well as advances in steel construction and new developments in the protection of iron and steel surfaces from corrosion, also were favorable to steel construction.

The Morse Type steel plant differs from the usual concrete plant in the shape and placement of basins, but the equipment used is the same as for concrete plants. A site for the New Britain plant was chosen on a rise of land above the dam and pumping station, in order to give an increased pressure to the city of about 27 psi. The basins are arranged in a circle around a central pipe gallery (see Figures 1 and 2), and all basins and equipment are in duplicate, so that repairs to any part will not necessitate shutting down the whole plant. The filter plant was constructed by the Chicago Bridge and Iron Co. and put into operation in 1940.

Water from the pumping station passes through a 30-inch C.I. pipe to the "Head House" (see Figure 3), which contains the offices, chemical laboratory, chemical feeders, heating plant and some chemical storage. At this point alum, and occasionally soda ash, is added to the raw water by means of chemical dry-feeders. The treated water is divided into two streams and flows to two Aeromix units, in parallel, installed in basins having a detention time of five minutes and thence to two paddle-mix basins of 20 minutes' detention time. The paddles,

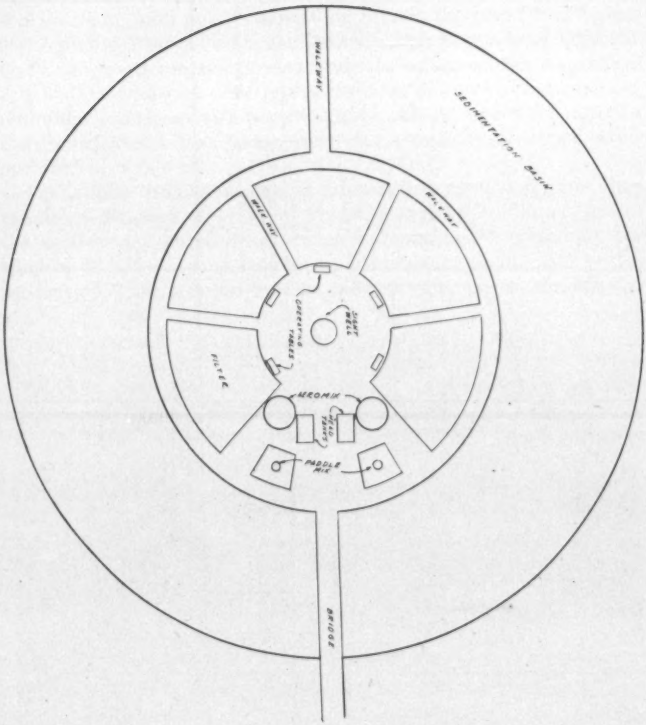


FIG. 1.—PLAN OF NEW BRITAIN FILTER PLANT

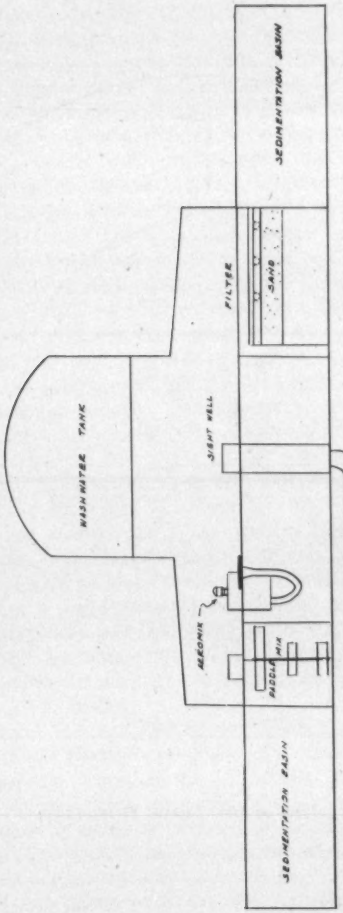


FIG. 2.—SECTION NEW BRITAIN FILTER PLANT

attached to a vertical shaft, rotate with a tip speed of about 1 ft per sec. The flocculated water then passes over weirs to duplicate coagulating basins, provided with baffles to prevent "short-circuiting" and insure uniform flow. The settling time in the coagulating basins is 4 hours at 10-mgd rate. The basins, which have a total depth of 16 ft, are unwatered and cleaned when the sludge reaches a depth of 8 ft at the outlet end. The settled water again flows over weirs to a downtake, connected with a central pipe gallery, where the water is distributed to five filter units, each with a capacity of 2 mgd when operated at a rate of 2 gal per sq ft per min.

The filter beds are somewhat unusual in construction. The filter medium, consisting of 18 in. of graded gravel and 24 in. of sand—the sand having an effective size of 0.57 mm and a uniformity coefficient of 1.65—is supported by a perforated steel plate with 11/16-in. holes 9½ in. on centers. The filtered water passes through the

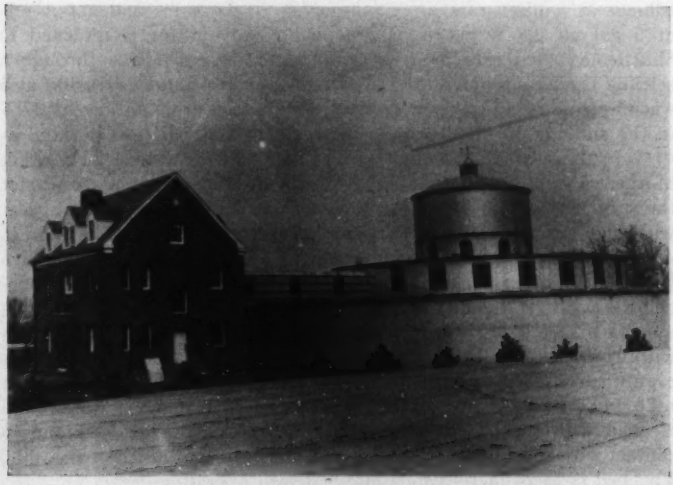


FIG. 3.—HEAD HOUSE AND FILTRATION PLANT, NEW BRITAIN, CONN.

perforated plates to a filtered-water chamber directly below the sand bed, and thence through rate controllers to a white tile-lined "Sight Well" in the very center of the plant, where the color and turbidity can be observed at any time. From here the water again passes to the Head House and an equalizing chamber, where lime, chlorine and sodium fluoride are added. The filtered and treated water is stored in two steel clear-water basins, each of 1.75-mil gal capacity. Because of the increased industrial use occasioned by defense production, the plant now operates a good portion of the time at a rate of 15 mgd, or 3 gal per sq ft of filter area per min.

Cleaning of the beds is accomplished by back-washing at a rate of 15 gal per sq ft per min with filtered water from a 100,000-gal storage tank, located on the top of the filter plant, above the operating floor. Filter runs of 75 to over 100 hours are frequent and, in spite of these long runs, the beds are usually effectively cleaned in a 5-min washing period. The wash water used amounts to about ½% of the effluent at a filter rate of 2 gal per sq ft per min, and about 1% at 3 gal per sq ft per min. The waste wash-water is returned to Shuttlemeadow reservoir by a canal, after having passed through a settling basin to remove the bulk of the sludge. Sand expansion at a washing rate of 15 gal per sq ft per min varies from as low as 27% in the summer to about 33% in the spring and fall, and as high as 41% in the winter. No sand has been lost from the beds as a result of the washing operations. Tests are made about once a month for the presence of mud balls, and the sand is treated about twice a year with high-strength calcium hypochlorite, in order to keep the sand in good condition and also to remove a small amount of manganese which has been found to accumulate on the sand. Fifteen to 25 lb of hypochlorite with 70% available chlorine are used for a single bed, having a surface area of 705 sq ft. The water in the bed is drawn down to within several inches of the sand surface, and the hypochlorite is evenly spread and mixed with the sand and water by use of a garden rake. The water is then drawn into the sand, so that the water level is even with the top of the sand. The bed is allowed to stand in contact with the chlorine for at least four hours, and longer if possible. The bed is then back-washed and put into operation.

Operation of the filter plant, which is on a 24-hour basis, is

controlled entirely by laboratory tests (see Fig. 4). The pH and alkalinity of the raw, coagulated and filtered waters are determined at least eight times daily, while the color and turbidity of the filtered water are noted every hour. Daily tests are made for temperature, threshold odor, hardness, residual alum, free carbon dioxide, and chlorine, while the fluoride content is determined three times a day —once on each shift. Floc tests, to determine the optimum amount of alum and soda ash, are made at least once a day. Frequent tests are made for algae in the reservoirs, bacteria in the raw and filtered

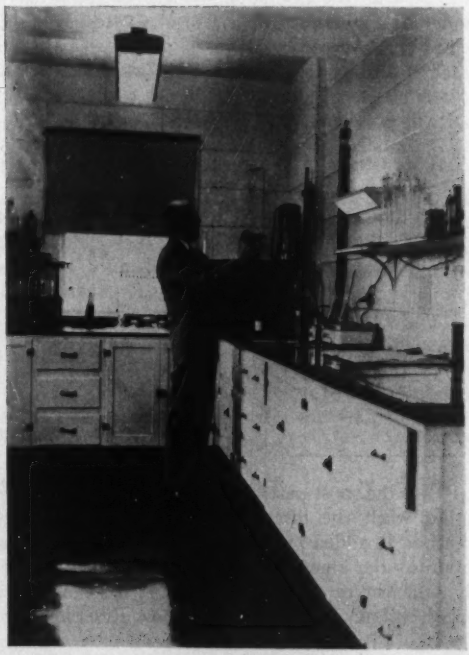


FIG. 4.—LABORATORY AT NEW BRITAIN FILTER PLANT

supply, depth of sludge in the coagulating basins, and mud balls in the filter sand. Occasional tests are made for manganese in the sand, and sanitary chemical and mineral analysis of the raw and filtered supplies.

Typical ranges for the various control tests are given in Table 2.

When the Shuttlemeadow supply only is used, there is sufficient alkalinity to provide coagulation of alum without the use of soda ash. The dosage range varies between 0.5 and 0.8 grains per gal to obtain a satisfactory floc. When a combination of Shuttlemeadow and Whigville is used as the raw supply, there is insufficient alkalinity present, and soda ash is used at a rate of 0.2 to 0.6 grains per gal. The alum may then vary from 0.6 to 1.2 grains per gal. When White Bridge water is also used in combination with Shuttlemeadow and Whigville, it is unnecessary to use soda ash and about 0.6 grain per gal of alum will provide a good floc.

TABLE 2.—TYPICAL RANGES OF RESULTS OF CONTROL TESTS OF NEW BRITAIN WATER

	Shuttlemeadow	Whigville	White Bridge	Filtered
pH	6.3 - 6.9 Average 6.7	5.7 - 6.1 Average 5.9	7.2	8.7
Color	10 - 55 Average 25	6 - 160 Average 15	0 - 5 Average 3	2
Turbidity	3 - 30	1 - 150	0	0
Alkalinity	7 - 14 Average 10	1 - 2 Average 1.1	25	14 - 15
Threshold Odor	2 - 24	2 - 30	—	0 - 4
Fluoride	—	—	—	1.0
Chlorine Residual	—	—	—	0.05 - 0.10

Odor is, for the most part, effectively controlled by the Aeromix machines, but when the threshold odor approaches a figure of 4, activated carbon is added by a dry-feed machine at the same point of application as the alum and soda ash. About 10 lb per mil gal are usually sufficient to control the odor. It has not been necessary to use copper sulfate in the Shuttlemeadow Reservoir for over ten years.

The filtered water is further treated with chlorine as an additional

safeguard for the bacterial quality, with lime to reduce the corrosive characteristics, and with sodium fluoride to reduce the incidence of dental caries in the children of New Britain. The chlorination is accomplished by means of the usual liquid-feed chlorination and chlorine is used at a rate of 3.5 lb per mil gal. Lime and sodium fluoride are added by means of dry-feed machines, the lime being added at the rate of 0.5 grain per gal.

A separate room was constructed in 1951 in the top floor of the Head House and, under the direction of George Wood, Chief Engineer for the Water Commission, equipment for ventilating the room and feeding the fluoride was installed. A Wallace & Tiernan dry-feed machine, Type A-635, is now in use and very little deviation from 1.0 ppm fluorine is found. Checks on the dosage rate are made not only by chemical tests previously noted, but also by continuous weighing of chemicals in the feeder. Tests made by the State Health Department Laboratory, as well as the city laboratory, also show the uniformity of the fluoride addition.

The maintenance of the plant has not been difficult, in spite of having been constituted of steel. The outside of the steel tanks has been re-painted only once since construction, about six or seven years ago. The original Bitumastic enamel is still in satisfactory condition, although a few minor repairs have been made. No repairs have been necessary to the solid cork insulation or the cork-paint insulation.

The author wishes to acknowledge the helpful suggestions of James A. Newlands, President, Henry Souther Engineering Co.; George W. Wood, Chief Engineer, New Britain Board of Water Commissioners; and Albert J. Murphy, Chief Operator of the New Britain Filter Plant.

THE DEVELOPMENT OF A LARGE WELL SUPPLY

BY FREDERICK B. SILLIMAN*

[*Read April 16, 1953.*]

In 1951, the Bridgeport Hydraulic Company, like many other water companies, was faced with the problem of securing additional water. Our immediate task was to produce at least 3 mgd additional supply for the eastern section of our territory, where a shortage was apparent.

Our first thought was to begin construction on a large storage reservoir. We had two potential sites and had been acquiring some property rights over a period of years. Detailed cost estimates for each site were made, and the results carefully evaluated. Either reservoir development would have cost in excess of $3,500,000 and furnished water at a cost of $140 to $150 per mil gal.

While the cost was relatively high, the time required to construct this type of project was also a factor. The company was faced with the problem of increased demands, due to industrial activity brought on by the defense effort. For example: The Avco Manufacturing Co., a large aircraft company, opened a factory and stated they would require 1½ mil gal of water each day. It then seemed that the only practical and immediate solution of our problem lay in the possibility of well fields.

In New England, generally speaking, large well fields are not common. The rocky formation of the ground, the presence of impurities detrimental to a water supply and, most important of all, the proximity of salt water are distinct disadvantages. In an area such as ours, the chances were great that one of these natural obstacles would be met. We were determined, however, with many reservations as to the possible outcome, to investigate the field.

In view of the urgency of meeting the demands placed upon us and our limited knowledge of ground water, we secured the services of a competent consulting engineer, Thomas H. Wiggin of New York

* Assistant to President, Bridgeport Hydraulic Co., 835 Main St., Bridgeport, Conn.

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City, and the well-construction firm of The Stephen B. Church Co., Seymour, Conn. With their assistance, a general survey of the surrounding area was made. To our dismay, only one location was found that offered any potential. This was 4 miles north of the city of Shelton and adjacent to the Housatonic River.

After securing permission of the property owners, 20 test wells, 6 and 8 in. in diameter, were driven. These were over an area of approximately 50 acres. Some indicated high water-producing ground, while others were of a silt nature the entire distance to rock—about 100 ft below the surface. As a result of these tests, the geological formation appeared to be a series of pockets, varying greatly in soil texture. The more promising wells were tested for yield with a small deep-well turbine pump, driven with a gasoline engine, and at least 3 sites proved adequate.

All this was fine, but to use the supply required considerable imagination. The water would have to be pumped over an adjacent rocky hillside onto the watershed of an existing distributing reservoir. This would present an operating head of 550 ft and the need for construction of a large-diameter pipeline up the 40° rocky slope. The whole picture warranted an economic study, to determine the feasibility of such a development. To a rather conservative utility like ourselves, the cost of utilizing a potential supply under such conditions was thought to be prohibitive. A conservative estimate of the capital cost and operating cost per mil gal was made. It showed clearly that the project was sound and that water could be produced at a reasonable price. Later in this paper, a breakdown of the capital and estimated operating costs is shown.

Three gravel-packed wells, spaced about 400 ft apart, were sunk. They are 3 ft in diameter and nearly 100 ft in depth. Each is equipped with Johnson Everdur bronze screens, the length depending on the soil texture and the water-yielding potential. Each well was subjected to a 30-day flow test, in which a small turbine pump, powered with an electric motor, was used. Upon completion of these tests, suitable permanent pumping equipment was designed, to produce a combined yield of 6 mgd.

A difficult engineering problem to be decided was the source of power to be used in forcing water from the wells over the adjacent

hillside. This involved a power requirement of 750 hp and consideration of diesel vs. electric motors. The electric rates in this area were relatively high and could have made the operation uneconomical. We therefore investigated the possibility of diesel power. This would have involved a diesel-powered centrifugal pump, to force water over the adjacent hillside, and a diesel generator set, to power small electric driven turbine pumps, to get water above the ground. The annual operating cost would have been satisfactory, but the capital outlay was substantial and the setup lacked flexibility, if and when the well field was more fully developed. The problem of electric power was complicated by the fact that we could not guarantee the electric company what our requirements would be during any one year. The development is basically for standby purposes, in the event of a dry season when the reservoir itself could not meet the demand. In a very wet season the amount of pumping would be nominal, but in a dry season the demand upon this system would be great. After considerable negotiations with the power company, an off-peak electric rate was agreed upon, which gave the electricity a definite edge over diesel equipment.

Under this agreement we can operate 463 hours out of each 720 hours per month, or 65% of the time. The basic off-peak energy rate is $.00874 per kilowatt-hour. There is also an annual carrying charge on the 20,000 ft of power line that was constructed for our sole use. The carrying charge continues for a five-year period and will be renegotiated at the end of this time.

The laying of a 30-in. pipeline, 9,000 ft in length, was a difficult construction job. A 700 ft section went through a bad swampy area that necessitated the driving of piles. A 1,000-ft section was on a rocky hillside having a 40° slope. Lock Joint pipe was installed, designed for pressures to meet operating conditions. The pipe-laying job was awarded to the A. Stanley Mundy Co., Woodbridge, N. J., who did an outstanding piece of work at a very reasonable cost.

The following tabulation gives an analysis of a sample of this well water, collected during a recent trial run.

AVERAGE ANALYSIS FOR HOUSATONIC WELLS SUPPLY

Supply	Well #1	Well #2	Well #3
Physical—Color in ppm	0.	0.	0.
Odor	0.	0.	0.
Turbidity in ppm	0.	0.	0.
Temperature, °F.	46.0°	46.0°	46.0°
Chemical—Chloride	5.04	3.36	4.92
Albuminoid Ammonia	0.010	0.005	0.005
Free Ammonia	0.010	0.005	0.000
Nitrite	0.000	0.000	0.000
Nitrate	0.160	0.070	0.160
Total Hardness as $CaCO_3$	22.5	19.0	21.
Alkalinity as Ca Co_3	13.5	13.5	15.
Free Co_2	11.	10.	11.
pH	5.7	5.9	5.8
Total Solids	43.	41.	51.
Fixed Solids	31.	25.	32.
Iron—Total	0.04	0.02	0.03
Bacteriological—No. of Samples	1	1	1
Gas in Lact. Broth, 24 Hr	0 0	0 0	0 0
Gas in Lact. Broth, 48 Hr	0 0	0 0	0 0
Coliform Organisms Confirmed	0 0	0 0	0 0
M.P.N. of Coliform per 100 ml	Less than 2.2	Less than 2.2	Less than 2.2

From the above, it is apparent that we are most fortunate in obtaining water of such high quality. The development is now completed to meet our current needs, with the provision for future expansion at little additional cost.

The capital cost and operating cost are of interest to us all. Below is a breakdown of these costs, together with a few notations as to our method. It will be noted that in these costs one of the very important factors is the cost of capital. For a private company this will average between 8 and 10%, depending upon the capital structure.

Provision has been made in the initial construction plan so that additional wells can be sunk, to increase the off-peak yield to at least 6 mgd. The work would not exceed $60,000 and, when realized, the annual cost per mil gal will be reduced to $58 per mil gal.

CAPITAL COST

Land and Rights-of-Way	$57,840
Large Wells and Test Wells	62,313
Pumps, Motors, Small-Diam. Piping, All Electric Equipment, Installation	111,998
Pump Houses	23,508
30" Pipeline	180,928
Engineering	20,207
Total	$456,794

ESTIMATED ANNUAL OPERATING COST

Based on 4.15 mgd yield for off-peak pumping, or 65% of 6.4 mgd potential

Cost Capital or Money[1]				
$456,794 @ 8.4%			=	$38,371
Local Taxes				7,500
Depreciation:[2]				
Wells	$62,313 @ 4% —	$2,493		
Pumps, etc.	$111,998 @ 3 1/3%	3,733		
Houses	$23,508 @ 2% —	470		
Pipeline	$180,928 @ 1% —	1,809	=	8,505
Energy[3]				
3,650,000 kwh @ $.00874			=	31,901
Cable Carrying Charge per Contract			=	10,440
Operation, Maintenance and Repairs			=	6,000
			Total	$102,717
Cost per Million Gallons—		102.717		
		4.15 x 365	=	$67.81

[1] Based on average cost of capital during 1951. Includes bond and bank interest, dividends on equity stock and applicable Federal taxes.

[2] Engineering cost to be distributed at later date and not included in above operating cost.

[3] Based on energy used during 10-day operational test.

It will be of interest to those concerned with the financial end of the water-works business that we were successful in an application for accelerated amortization on this project. Application was made to the Dept. of Commerce, Division of National Production Authority. We are allowed to write off in five years 45% on the adjusted cost (actual cost, less land, rights-of-way, test wells, and engineering), which is equivalent to 71% of the actual cost.

As a result of our experience in securing a large ground-water supply, we have reached a few conclusions:

1. Secure the services of a consulting engineer who has had considerable experience in this field.

2. The ultimate yield of a well can best be determined by a 30-day continuous test. This should be carried out on the test well and on the final well.

3. It must be remembered that a new well development is a calculated risk, regardless of what anyone says, and you should proceed with caution at all times.

NEW ADDITIONS TO WEST PARISH FILTERS

BY PETER C. KARALEKAS*

[*Read May 21, 1953.*]

Little River Supply

The development of the Little River watershed—a high-elevation (average of 1,300 ft above sea level), sparsely settled catchment area—as an additional source of supply for the city of Springfield was begun in 1906. The first water entered Springfield in December, 1909. The works consisted essentially of Borden Brook Reservoir, the "Intake" reservoir, a mile of tunnel, the raw-water sedimentation basin, six filters at West Parish, a filtered-water storage reservoir on Provin Mountain in Agawam, a single 42-inch-diameter conduit 7.5 miles long between West Parish Filters and Provin Mountain Reservoir, and the 42-inch-diameter transmission line 4½ miles long from Provin Mountain Reservoir to the city. Four filters and another conduit from Provin Mountain Reservoir to the city were added in 1924 and 1928, respectively. To insure a dependable supply of larger capacity, the Cobble Mountain impounding reservoir, diversion tunnel, pressure tunnel and hydro-electric plant with accessories were constructed between 1927 and 1930. Another section was added to Provin Mountain Reservoir for filtered-water storage in 1931-1932. Two and one-half miles of the second main from West Parish Filters to Provin Mountain were installed in 1943 and the remaining 4.9 miles were completed in 1949. In 1951-1952, four more slow-sand filter units were added.

The Cobble Mountain Reservoir has a storage capacity of 22.829 billion gallons, with a watershed area of 45.8 sq mi, nearly 40% of which is owned by the city of Springfield.

The dependable yield of the Little River Supply is 55 mgd. The water is supplied to the city entirely by gravity.

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West Parish Filters

The West Parish Filters provide the only treatment to the Little River water. The original filters consisted of six covered rectangular slow-sand filters, placed side by side, each having an area of 0.52 acres. In 1924, four more units, each 0.82 acres, were added, making a total area of 6.40 acres. The newer units, placed in pairs and opposing, with a regulator house with enclosed sand washers in between, were better balanced hydraulically and provided superior working conditions for the personnel, particularly for winter operations.

By 1950, the water consumption had increased to an average of more than 25 mgd, with a weekly peak exceeding 34 mgd. These rates placed a demand upon the filters greater than they could be expected to satisfy with any degree of safety.

The engineer's report on these conditions stated, "The average daily output for 1949 was 25.85 million gallons, which according to our observations was producible only by extremely good operation procedures and fortuitous relief from breakdowns and other emergencies. It will be noted that the average filtration rate, assuming all units were in service, exceeded 5 million gallons per acre per day. Under practical operating conditions, rates considerably in excess of 5.0 million gallons per acre per day were frequently required to satisfy the demands."

Because of the high demands on the plant, none of the units could be taken from service, except for the most urgent reasons. We were forced in the interests of effective administration to undertake the installation of the new filters. And it might be added that within one week, July 17th to 24th, after the new filters were put in service, the peak consumption reached 38.5 mgd for a few days—a demand which we should have been hard pressed to meet without them.

Slow-Sand vs. Rapid-Sand Filters

Several people, particularly those not familiar with water-works practice in New England, have posed the question as to why the "old-fashioned slow-sand filters" were installed in preference to the "modern rapid-sand filters". The answer to that question is that the

cost of processing a mountain water like Little River is less with slow-sand filters. Before it was decided to construct the additional filters, an analysis was made of our actual treatment costs, which indicated that the total operating cost for the existing slow-sand filters was less than would be the cost of chemicals alone for rapid-sand filtration. A controlling argument for rapid filters is a saving in land areas of about 90%. This consideration does not control at West Parish, where the city owned sufficient land for either type of filters. Furthermore, since extensive modifications would have to be made for coagulating and settling, the usual differentials in installation costs in favor of rapid-sand filters did not exist. The city of Springfield owns about 40% of the watershed and adequate control of pollution on the entire watershed results in an unusually good quality of water, which has satisfied all of the standards for potability after treatment by slow-sand filters only.

Their simplicity, as well as the proven methods of operation which the operating personnel were used to, lent further weight to the preference for slow-sand filters for the additional units. We maintain, therefore, that our new slow-sand filter plant is not "old-fashioned". It is a modern water-treatment plant, designed on old and tried principles, equipped with new appurtenances, simply controlled, requiring a limited number of trained personnel, efficient, and producing a water which meets all sanitary standards—as good in quality as any produced in the world.

Size, Location and Arrangement of the Beds

Although the city owns ample area to provide filtering capacity for the entire dependable yield of Little River, the immediate and probable future demand would not warrant such an increase in capacity at one time. Two plots, each sufficient for approximately two acres of slow-sand filters, were available for the needed expansion. The site adjacent to the original filters was selected as being more economical for present construction. Further, the excess of earth excavated over that needed could all be used to bring the second site up to required grade, which not only would provide a short haul for disposal of the excavated material, but would reduce the cost of additional future filters, if or when required.

The number and arrangement of the filter beds were carefully studied with respect to the operating experiences with the existing filters, to the hydraulic requirements of the site, and to the construction cost. The gravity discharge of waste water, as well as filtered water drainage from the beds, made it necessary to locate the filter beds as high as possible. The maximum elevation of the beds, on the other hand, was limited by the low-water levels in the sedimentation basin and intake reservoir. Because of these limitations, it was necessary to establish the elevation of the new beds about eleven feet above the existing beds.

The importance of this new elevation with respect to the selection of the number of beds is readily appreciated, since at least three beds of the four must be operable as a unit—one bed inoperative for extended repairs or resanding, one for scraping or raking, and one in operation for refilling the scraped or raked bed. This minimum of three beds in a unit provides for no delivery to the city during the refilling period. Although there are certain cost factors which increase as the number of beds increases, the symmetry obtainable with four beds tends to compensate for them costwise in other details and permits better hydraulic balance.

It was decided to construct four beds, arranged in pairs, with the regulator house between, rather than four in a row, for the following reasons:

1. Central control of all operations makes for easier supervision of labor and inspection of equipment.
2. Better architectural treatment is possible for a single large structure than for four widely scattered smaller structures.
3. Construction and maintenance of regulating services are facilitated.
4. There is less piping and a lower maintenance cost of piping.
5. Maintenance and operation in inclement weather are easier.

Design Details

It is not the purpose of this paper to describe minutely all of the details of the design nor many of them. Summarily, the usual number (whatever that might be) of problems, inherent in adding to existing structures, was encountered in this instance. Certain departures from

both the earlier designs were made possible through changes in distribution of construction costs in recent years between material and labor, through improved technique, and through operation experience over a period of a half century. The principal items in each of these categories are of particular interest:

1. *Groined-Arch vs. Reinforced-Concrete Construction.* Groined arches were used in the earlier filters. Several types of structures with variations in panel size were investigated in detail. Beam, slab and girder construction in the panel sizes required for filters is the most expensive and least adaptable for filter construction. Large-span panels, 21 ft and over on a side, with flat roof slab with drops at column, result in expensive construction and are not particularly adaptable for the conditions. Fifteen-foot panels, which involve an increase of 1½ ft over the column spacings in the existing filters, are the most economical panel size for the areas available. Four types of construction were analyzed in detail for this panel size: (a) flat-slab roof and flat-slab bottom, (b) groined-arch roof and bottom, (c) slab-and-beam roof and bottom, and (d) a combination of flat-slab roof with ridge-and-valley bottom. The last type—(d)—combines the best and most economical features of the two types suited for filter construction, and eliminates many of the objectionable features of groined arches, the economy of which has changed in the past 20 years because of (a) the higher costs for forms, (b) the higher costs of labor and hauling, and the labor costs of handling of the selected backfill, which must be placed against the vertical walls to resist the thrusts of the end arches, and (c) the higher costs of placing fill over the arches. Type (d), a combination of flat-slab roof with ridge-and-valley bottom, proved the least expensive and provided the best hydraulic flow conditions.

2. *Rate Controllers.* Because of the lower rates of filtration through slow-sand filters, the rate of change of filtration is not as great as in rapid-sand filters. On the existing filters, the rates are adjusted by throttling the effluent valves manually—a trial-and-error process, requiring time and experience. Automatic rate controllers were installed on the effluent lines of the new filters, to lessen routine operating time, take advantage of the improved operating technique, and provide closer control.

3. *High-Capacity Bottoms.* Experience in operating the existing filters demonstrated that filtering rates approaching 10 mil gal per acre per day had resulted in no demonstrable effect on the quality of the water. The loss in head in the collecting systems of the existing filters, however, prevented their operation at higher rates or for extended periods. It was desirable, therefore, to investigate higher-capacity filter bottoms of a type where distribution would not be adversely affected and the cost would not be prohibitive. Our extended study of the distribution indicated that for the spacing of the underdrains selected, theoretically there should be no appreciable difference in filtering rate over the entire filter area. Differences in sand grading would have a greater effect on rate distribution than the average filtering rate over the entire area. After it had been concluded that the distribution would not be adversely affected with the underdrains spaced according to the dictates of structural economy, hydraulic design of the drains was routine. The estimated additional cost of $2,500 to $3,000, because of greater slopes and larger conduit sizes, was deemed well worth the opportunity to demonstrate under full-scale conditions that filtering rates up to 15 mil gal per acre per day could be safely used for Springfield water.

It might be added that, as yet, we have not attempted rates as high as 15 mil gal per acre per day. We have, however, operated two of the four filters at a rate of 14 mil gal per acre per day for several days, with apparent good results. One observation which was made indicated that the filter must be in good clean shape before this high rate can be achieved.

Construction Problems

1. *Trench by Existing Filters.* In general, there were no unforeseen problems in the construction of the new filters. As with all projects involving additions to old plants, careful consideration had to be given to safeguarding the existing structures. In this instance, a drain and an effluent pipe passed within a few feet of the original filters. These filters are of groined-arch construction, with little or no reinforcement, the horizontal thrust of the arches on the side walls being taken by special fill material, carefully placed and compacted as the walls were built up. To safeguard these old

walls, heavy wood sheathing and bracing was specified. The contractor, however, elected to drive H-column piles, 4 ft on centers, in a line roughly parallel to the filter walls, to a depth of approximately 4 ft below the filter bottom. As excavation of the wide trench for the two pipelines proceeded, planking was placed between the columns and heavy bracing from the columns across the trench was installed after the new pipes were placed and tested. New fill was obtained and particular care was used in placing and compacting it against the sheathing. The sheathing and H-column were left in place. No distortion of the existing filters has been noticed.

2. *Placing Concrete.* It is of further interest to note that all of the concrete in the new filter construction was pumped to the forms. It was noted early in the project that the slump of the concrete entering the forms was considerably less than that entering the pump. Since the maximum distance pumped did not exceed 400 ft, this decrease in slump would not be caused by the time factor only. It was concluded that the heat of friction in the pipe probably hastened the reaction sufficiently to effect the slump. This was adjusted by increasing the proportion of cement by ¼ bag per cu yd, in order to maintain the strength and workability.

(Ready-mix concrete was used throughout. Test cylinders of the concrete as placed showed a uniform compressive strength of the concrete between 200 lb per sq in. and 400 lb per sq in. above that specified, 3,500 lb per sq in. Only a few cylinders broke outside of this range. The finished concrete was free of honeycomb and otherwise demonstrated the uniformity of the product, as well as the excellence of the work.)

3. *Placing of Filter Sand.* After sand was placed on the existing filters, the filters were placed in operation and permitted to filter to waste, until the absence of coliform organisms in all 5 of the 10 ml portions of daily samples was consistent. This filter-to-waste period varied from six to eight weeks and was considered, according to the old theories, as seasoning. Regardless of these theories, it appeared obvious that the sand and gravel as delivered would unquestionably harbor numerous coliform organisms and, if these were killed, this seasoning period might be reduced. Accordingly, the following detailed specifications were given as to the placing, as well as the physical characteristics, of the sand:

"*Material.* The filter sand shall be washed and graded and shall consist of hard, sharp, uncoated siliceous grains free from clay, loam, dirt and other deleterious substances with not more than 1% flat or micaceous particles. The sand shall not contain more than 2% calcium and magnesium taken together and calculated as calcium carbonate; and not more than 2% by weight shall be lost on ignition of the crushed and powdered sand.

"The sand shall have an effective size not less than 0.25 nor more than 0.35 mm. and a uniformity coefficient of not more than 3.0. The sand shall be free from dust and shall not contain more than 1% finer than 0.13 mm., and shall be entirely free from particles larger than 5.0 mm.

"*Sterilization.* The inside of all pipes and surfaces of other structures which are to be in contact with the water shall be smooth and free of extraneous projections and brushed or scrubbed clean of all dirt and foreign substances. After cleaning, these surfaces shall be sterilized by contact for a period of 24 hours with water having a free chlorine content of at least 100 ppm. or by other approved means.

"The filter sand shall be sterilized in the sand bins before *it is placed in the filters,* in the following manner. The bin into which sand is to be placed shall first be half-filled with water having a free chlorine concentration of at least 125 ppm. The sand shall be dumped slowly to promote dispersion through the water and settlement in uniform layers under water. As the bin is filled with sand, the water level shall be maintained at least one foot above the sand and a concentration of not less than 100 ppm. of free chlorine shall be maintained in the water. The sand and chlorinated water shall be maintained in contact for at least 24 hours before the sand is placed in the filters.

"If in the opinion of the Engineer, the gravel is not adequately chlorinated by the chlorinated water used in hydraulically placing the sand, approved methods shall be employed to assure sterilization of the gravel.

"*Installation.* The sand shall be transported hydraulically from the bins to the distributors, and shall be placed in the filters to the lines shown on the drawings. The sand shall be placed in three layers, the first approximately 1 foot 6 inches thick, and the other two

approximately 1 foot thick, so that the ultimate thickness after compaction will be 42 inches. Care should be taken to prevent disturbing the gravel during the placing of the sand. Each layer shall be raked level before the next layer is placed. During the placing of the last foot of sand, a layer of water 2 to 4 inches deep shall be maintained over the top of the sand, in accordance with instructions from the Engineer. The top of the sand after final placing shall be exactly level and parallel to the water surface."

Prior to the placing of the sand, it became apparent that time would not permit the hydraulic transmitting of all the sand from the bins to the filters, if the filters were to be available for the high demands of the summer of 1952. It was agreed, therefore:

1. that the chlorinated sand would be raised mechanically from the bins and transmitted hydraulically by non-chlorinated water to the separators, placed over the manholes in the top of the filters, and

2. that all three layers of sand would be placed with the drains open—hence, not under water; levelling was controlled by marks on the walls and columns; and

3. that after the third layer was so placed and levelled, and before chlorinating, the sand was to be flooded through the underdrains, drained, inspected, and relevelled if necessary, and

4. that after all of the sand was placed, levelled, flooded and drained, the filter would again be filled through the underdrains with water having sufficient chlorine to produce a residual of at least 25 ppm after it had passed upward through the filter medium. The filter so filled with chlorinated water was not to be drained for at least 24 hours.

Completion of the Project

Work was started in January, 1951, and except for seeding the area and items of maintenance, was completed on August 1, 1952. Inclement weather and difficulty in obtaining scarce materials—particularly reinforcing steel—tended to retard progress, but did not result in a complete work stoppage. The first filter was completed and sterilized in the first week of May, 1952, and filtered water from it was delivered for consumption on June 26, 1952. By July 17, 1952, all four of the filters were in service.

The project involved the demolition of an existing garage and its replacement by a new, fully equipped, two-story garage; the relocation and the rubble lining of the stream; the building of one new bridge and alterations to an existing bridge; installation of new septic tanks and disposal fields, and the modification of drains.

The entire project cost $1,127,513.51 and required approximately:

(a) 60,000 cu yd of excavation;
(b) 15,000 cu yd of concrete;
(c) 400 tons of reinforcing steel;
(d) 1 mile of split-tile underdrain;
(e) 1,800 cu yd of filter gravel;
(f) 11,500 cu yd of filter sand.

The general contractor was Ley Construction Co. of Springfield, Mass. The filters were designed and the construction was supervised by Bogert-Childs Engineering Associates of New York, N. Y. The testing and inspection of materials and concrete sampling were done by the Jersey Testing Laboratories of Newark, N. J.

QUALITY CONTROL OF THE WATER IN THE SPRINGFIELD SYSTEM

BY JOHN P. LYNCH*

[*Read May 21, 1953.*]

Because of the presence of many impurities in natural waters, purification methods must be utilized to make it safe and potable for its consumers. These treatment methods may be grouped, according to their accomplishment, under a relatively small number of general headings, which are as follows:

1. Removal of floating matter, generally by screening.
2. Removal of suspended solids and color-causing matter by:
   a. Sedimentation
   b. Coagulation
   c. Filtration
3. Removal of bacteria by disinfection with chemical agents or by filtration.
4. Removal or neutralization of tastes, odors, objectionable minerals, and dissolved gases by:
   a. Aeration
   b. Treatment with chemicals
   c. Special equipment and methods, such as ozonation.
5. Removal of hardness by softening.

In the Springfield system we use screening for the removal of floating matter, sedimentation and filtration for the removal of suspended solids, filtration for the removal of bacteria, and aeration for the removal of dissolved gases.

These methods were adopted as being most practical, due to the natural advantages we have in Cobble Mountain Reservoir, which is one of the largest reservoirs in New England. It is a large, deep reservoir with steep sides and has a sparsely settled watershed area. It impounds 22.5 billion gallons of water. This water is of a very high quality, both chemically and bacteriologically, due to natural purification processes.

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This reservoir impounds enough water to take care of the needs of Springfield for 750 days. However, since the operation of the power house uses large quantities of water, the average retention period during which natural purification may take place is roughly one year.

Another factor in the purity of the water in Cobble Mountain Reservoir is the fact that the watershed contains many large areas which are uninhabited. The night soil from the few homes, which are in a position directly to pollute the reservoir, is collected by water-department personnel and taken to the sewage treatment plant of the city of Springfield at Bondi's Island.

In this system, we have very little trouble with tastes and odors. Occasionally during the summer we find algae growths, which we eliminate with copper sulphate. There is only a very slight possibility that these growths could cause us trouble, even if we did not treat them with algicides, because the water is drawn from the reservoir 120 ft below the surface. The temperature at this depth seldom exceeds 56° F., even during the hottest part of the summer. As is well known, algae will not grow well at this low temperature or at that depth, because of the absence of sunlight.

Because Cobble Mountain Reservoir is subjected to a strong wind action, a mechanical destruction of algae is accomplished on many days when ordinarily we should expect trouble with these microscopic growths. The organism which we find most frequently and in a quantity requiring treatment is *Anabaena*.

In the list of methods used by this department coagulation was not mentioned, because we discontinued the use of coagulants five years ago. With the exception of 5 or 6 days in the year, the color and turbidity are not sufficiently high to require this treatment. The average true color of the raw water in 1952 was 17 ppm, while the average turbidity for the same period was 3 ppm. On those days when storms occur, the source of our trouble is in that area of our watershed near the intake reservoir. This area is approximately 2.5 sq mi in size and is in the form of a gorge with very steep sides. When storms occur and scour this area, matter causing color and turbidity is washed into the intake reservoir. When these components are increased to the point where coagulation would normally be required,

we eliminate this need by closing the influent gates to the sedimentation basin and using this basin as our source of raw water. This basin holds 44 mil gal, of which we feel we can use 30 mil without stirring up the bottom when we refill. This means we can run approximately one day with the influent gates to the plant closed. When the storm has abated, we ask the power house to run 50 or 60 mil gal of Cobble Mountain Reservoir water through the intake reservoir to flush it out. The capacity of this intake basin is 35 mil gal, of which 20 mil are available for use.

The water in Cobble Mountain Reservoir is low in color and turbidity, even during storms, since for the past five years we have not had a storm of sufficient intensity to stir up this reservoir, due to its size and depth, and the volume of water impounded. When the turbid and highly colored water in the intake reservoir has either been flushed over the dam and down the river, or been diluted by clear water from Cobble Mountain Reservoir, the influent gates to the filtration plant are opened and the sedimentation basin is refilled. We find this method is more desirable, because the storms which cause us trouble generally come late in the year, when the temperature of the water is low. Generally, it is in the middle or high thirties and at this temperature a good floc would be difficult to obtain. Moreover, since our sedimentation basin is "short-circuited", the effective settling time is of short duration. This basin is shaped more or less like a four-leaf clover, with the influent water entering in one of the upper leaves and following its edge around to the stem and out of the basin. Because of these two factors, some of the coagulants would be carried over into the filters and the filter runs shortened. In the past, we did add alum and lime, using the influent tunnel to the sedimentation basin as a mixing chamber, and we were more or less successful, since the use of water was much less than it is today. Even with the "short-circuited" basin, the retention period was sufficiently long for adequate floc formation and sedimentation. Also, this method of bypassing the highly colored and turbid water saves us a considerable sum of money every year.

We do not use chlorine or any other chemical disinfectant, because we get an excellent bacterial removal by the slow-sand filters. The bacterial content of our raw water is very low. During 1952,

the average 20° C. standard plate count of the raw water was 250 colonies per ml. The average for this water after filtration was 13 colonies per ml. The average 37° C. standard plate count of the raw water was 45 colonies per ml and after filtration the average 37° C. standard plate count was 4 colonies per ml. The test for coliforms in the raw and filtered water gave the following results: 54.3% of 1,830 10-ml portions of the raw water confirmed positive for coliforms, while only 0.32% of a like number of 10-ml portions of the filtered water confirmed positive.

It may be of interest that Springfield is reported to be the only municipal surface-water supply in the United States which is not chlorinated.

The average for a 20-year period from 1933 to 1952, for the 20° C. standard plate count was 170 colonies per ml of the raw water; the 20° C. standard plate count after filtraton was 7 colonies. While maximum allowable standards have not been established for a 20° C. standard plate count, the counts we obtain would lead all authorities to give the water a high rating.

The 20-year averages for the 37° C. standard plate count were as follows: 17 colonies per ml in the raw water and 0.6 colonies per ml for the filtered water. The standard set by the United States Public Health Service is that the 37° C. plate count should not exceed 100 colonies per ml.

The chemical content of this water is also well within the limits set by the U. S. Public Health Service. The average results for the 20-year period from 1933 to 1952, inclusive, for our filtered water were: color, 12 ppm; turbidity, 0 ppm; hardness, 11 ppm; iron, 0.09 ppm; manganese, 0.00 ppm, and pH, 6.5.

At Provin Mountain, which is located approximately halfway between the filtration plant and the city of Springfield, we have a covered concrete storage basin, which holds 29 mil gal of water. Its purpose is to maintain an even pressure under varying water demands by the city and adjoining towns, which are supplied by this system. The water is admitted to this basin through an aeration network of 264 nozzles, which throws the water up into the air to a maximum of 22 ft. This aeration is desirable, because in this process the free carbon dioxide is swept out, absorbed by the atmosphere, and partially

replaced with oxygen, and as a result the pH is raised from 6.1 to 6.7; thus the water is made slightly less corrosive. Studies of the calcium-carbonate stability using Langelier's index show that less lime would be needed to stabilize the water after aeration than before aeration.

There has been placed in the area under the aerators a wooden frame, upon which was placed a layer of marble chips. The original idea is said to have been that as the aerated water came into contact with the marble, some of the lime in the marble chips would be dissolved by the water, thereby tending to stabilize it. It is questionable how effective this was when first used, since the contact time is extremely short. We do know that the water is not picking up any lime now, but we found that the marble tends to break up the drops of water and more surface area of water is available for adsorption. We also found that these marble chips provide shade over the covered area, which helps to eliminate algae growths, where the ponding of water occurs before entering the covered basin. On one occasion, these chips were removed from one section of the aerator, in order to repair the frame, and in a couple of days a growth of *Ulothrix* was found in the uncovered section. After cleaning the area and replacing the marble chips, no further trouble was encountered.

However, this aeration process has its disadvantages. During the summer months, we believe we are picking up a contamination from air-borne dust. The section containing the aerators is enclosed by a link-wire fence, topped with barbed wire. The gate is always kept locked, so that people or animals cannot approach within 30 ft of the aerator nozzles. Therefore, contamination from this source can be ruled out. The marble chips and scrapings from the frames holding them were checked and found to be negative for seeding by coliforms.

Prior to the spring of 1949, before a second pipeline between the filters and the Provin Mountain Reservoir was completed, the aerators were not used during the summer months, because it was found that the friction loss in the aerators was sufficient to reduce the output of the plant by 4 mgd.

During August of 1949, the samples of the effluent from this basin, which are collected daily, showed the presence of coliform

organisms. Therefore, we started collecting samples of the influent water, which were negative. There was a distinct difference in the appearance of the lactose-broth tubes after incubation. The tubes containing the influent water were clear, while the tubes containing the effluent water were very turbid and a heavy pellicle formed on top of the broth. During the period from August 13th to August 26th, the percentage of positive confirmatory tests was 0% for the influent water and 13.3% for the effluent water. We theorized that because this reservoir was situated about 250 ft in elevation above the farmlands of the area, dust was being blown up the sides of the hill and over the top of the reservoir, and was being washed out of the air by the aeration process. At times, heavy clouds of dust can be seen being blown across the farmlands and up the sides of the hill. As soon as the aerators were shut off and the water admitted directly to the reservoir, the samples of the effluent improved. In the 35-day period following August 26th, we had 11 positive tubes out of 1,030 10-ml portions, for a percentage of 1.1%.

For a three-month period following this with the aerators in use, we analyzed all positive tubes, using the I.M.V.I.C. differentiation tests. The results we obtained agreed percentage-wise with the results of other experimentors, who examined farm soils for bacterial types making up the coliform group.

We feel that we have very good laboratory control over the water in our system. Samples are collected daily from representative points on the system, including the distribution lines in the city. Bacteriological samples are obtained of the water entering and leaving the sedimentation basin; the effluents of each of the 14 individual filters; a composite sample of filters 1 to 6, filters 7 to 10, and filters 11 to 14; and a composite sample of all 14 filters. The water entering and leaving the storage basin at Provin Mountain is checked daily. According to U. S. P. H. S. standards for a city the size of Springfield, 140 samples per month should be collected. We collect about 175 samples per month, collections being made on all seven days of the week. Periodically, we make watershed inspections, collecting samples of the tributaries to the various reservoirs in our system. Chemical samples of the water entering the sedimentation basin, the water entering the filters, and the water leaving the filtration plant are

analyzed daily, including Saturdays and Sundays. Weekly samples are analyzed from Provin Mountain Reservoir. Samples from various points on the watershed and from the city are checked periodically for chemical analyses.

Approximately 100 microscopic analyses are made per year. Whenever a filter is resanded, 25 to 35 samples of sand are analyzed mechanically. During 1952, 14,007 samples were collected and analyzed in the laboratory for one thing or another.

In conclusion, Springfield's fine system depends mainly on the size and location of its Cobble Mountain storage reservoir. The raw water with which we work is of a high quality and requires a minimum of treatment, and, as a result, the city of Springfield and customer towns drink the best water available.

ODORS AND TASTE IN PUBLIC WATER SUPPLIES

BY DAVID H. TURCOTTE*

[*Read June 18, 1953.*]

The scope of this paper is not to propound any new revolutionary method to control odors and taste in public water supplies, but rather to reorient those already familiar with accepted methods and for those water departmental and public health officials not familiar with these problems.

Of all the physical characteristics of a drinking water, odor and taste are least tolerated by the public. These two properties are very closely related and often the sensation spoken of as taste is in reality perceived as odor. Chemically pure water is free from odor and taste. Most of the bad taste observed in drinking water is due not to inorganic compounds, but to organic substances in solution and suspension or to microscopic organisms.

Public water supplies are obtained either from ground-water sources or from surface-water sources, such as rivers, lakes or ponds. Both supplies are dependent for their volume upon the amount of precipitation.

Ground Water

Water obtained from the ground is usually free from odors and taste, but the author's experience with ground water at Lowell, Mass., from 1943 to 1950 proved otherwise. The bulk of Lowell's water supply at that time was obtained from wells, situated on the banks of the Merrimack River. This well field has been in continuous operation since 1895 and originally contained 500 2 1/2 in. tubular wells, having an average depth of 35 ft. The water pumped from these wells had an astringent taste and a disagreeable odor of hydrogen sulphide. The water was devoid of oxygen and contained 30 ppm of carbon dioxide and 4 ppm of hydrogen sulphide. The soluble-iron content of the raw water in 1943 was 3.6 ppm and by 1950 it had

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risen to 15 ppm. A contributing factor of this rapid increase was extended droughts in 1949 and 1950, which produced a corresponding drop in the ground-water table.

The volatile gases of hydrogen sulphide and carbon dioxide are swept out of the water by spraying and aerating the raw water on coke beds. These coke beds are 10 ft deep and act as trickling filters. The water which originally was devoid of oxygen has now reached 65% of saturation. Previously oxidized iron on the coke acts catalytically to bring about a 25% reduction of the iron content of the raw water, as it passes through the coke filters. Sedimentation and slow-sand filtration bring about the removal of the remaining astringent taste.

Ground waters as a rule are free from algae and bacteria. Tests made on individual wells at Lowell showed the presence of iron and sulphur bacteria. These organisms are contributing factors in producing bad odors and unpleasant taste. To eliminate growths of *Crenothrix* (one of the higher bacteria) in the wells, high-test hypochlorite and Calgon were added, and this mixture was allowed to remain in the inactive wells for 48 hours. The wells were then pumped free of residual chlorine, with the elimination of the sulphur and iron bacteria.

Surface Waters

Odor and taste are often present in surface supplies. These waters are often teeming with life of great variety of species. Many of the free-floating microscopic organisms or plankton, however, are of great importance, because their development in water reservoirs imparts objectionable taste and odors to the water.

The organisms, encountered in surface waters, which are significant in the control of water supplies may be grouped in the following classifications:

Plants

Bacteria—fungi, algae, and rooted aquatic vegetation.

Animals

Protozoa or single-cell animals
Rotifera or many-cell animals
Crustacea and insect larvae.

Microörganisms in Surface Supplies

The odor- and taste-producing organisms in surface supplies are usually free-floating plants or animals. They are the algae and protozoa.

The algae are flowerless plants of simple structure without mycelia, roots, stems or leaves. The protozoa are the lowest organisms belonging to the animal kingdom. The protozoan cell is a single mass of protoplasm that possesses in a general way all the properties of the protoplasm of higher animal cells. The protozoan cell absorbs its food at any point on its surface. Digestion takes place within the cell.

The algae obtain their food materials from the soluble nitrogen, phosphorus, potassium and other compounds present in surface waters. The amount of these food materials necessary to sustain growth is not large, although, as may be expected, abundant food material may mean abundant algae growth.

The algae and the protozoa owe their ability to produce odors and taste to the volatile oils which they are able to produce. The intensity of the odor and taste is dependent on the number of organisms present in the water.

Decomposition of Organic Matter

Odors and taste are also produced in surface supplies by the decomposition of vegetable and other organic matter. The complex organic compounds are split, with the production of gas and humus matter.

Odors and taste problems have been experienced in the Cambridge water. On July 15, 1931, a faint cucumber taste was detected at the filter plant. Microscopic examination showed that *Synura,* one of the flagellated protozoa, was present in one of the reservoirs. This reservoir was shut off and the supply treated with 4.5 lb of copper sulphate per mil gal of water. Three days later the reservoir was returned to service, free from the *Synura* organisms. In the winter of 1952 these organisms again made their appearance, accompanied by *Dinobryon,* another member of the protozoan family, which produces a fishy taste in the water. On August 10, 1952, the Cambridge supply had a mouldy and grassy odor. This trouble was due to the presence of 250 standard units of *Anabaena* (in heavy

concentrations this organism produces a pigpen odor), a member of the blue-green algae group of organisms. In general, these organisms are not troublesome, unless the water temperature is about 70° F. They are usually found within 10 to 15 ft of the surface of reservoirs, and may collect as a scum on the surface and cause the so-called water bloom after periods of stagnation and high temperature.

Conclusion

Since the growth of microscopic organisms is generally the cause for odors and taste in surface supplies, both mechanical and chemical means are successfully applied to inhibit or eliminate their growth.

Mechanical methods used to inhibit the growth of plants and microscopic life may be secured by stripping the top soil from newly-made storage reservoirs. The enacting of laws, preventing the discharge of industrial waste into streams and rivers, will also aid in controlling growths of the larger plants and microörganisms, by eliminating the vital pabulum which this industrial waste contains. The cutting-down of vegetable growths on sides and shores of reservoirs will also eliminate necessary pabulum, so vital for their growth.

The odors and taste present in water-supply systems may often successfully be removed by chemical compounds. Copper sulphate, activated carbon, black alum, high-test hypochlorite, and ammonia have all been successfully applied to improve the taste and odors in public water systems by destruction of microscopic life.

References

Whipple, George C., Fair, Gordon M., and Whipple, Melville C. The Microscopy of Drinking Water. John Wiley & Sons, New York (4th Ed., 1927).

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Cox, Charles R. Water Purification. Case, Sheppard, Mann, New York.

MEMOIR OF STUART E. COBURN

Stuart E. Coburn, member of the New England Water Works Association since 1941, chief chemist of Metcalf & Eddy, died on August 21, 1953, after a long illness. He was born in Walpole, Mass., August 8, 1890, the son of Edward B. and Elizabeth (Woodburn) Coburn.

Coburn graduated from the chemistry course at Worcester Polytechnic Institute in 1913. For a year after graduation he worked as a chemist at the Worcester Sewage Treatment Plant. From 1914 until his death he was employed by Metcalf & Eddy, first as a chemist and then as chief chemist.

Coburn's professional activities were related largely to problems of water purification and treatment of sewage and industrial wastes. His work took him to many municipalities throughout the United States and as far off as Hawaii.

He was a member of the American Chemical Society, American Water Works Association, and Federation of Sewage and Industrial Wastes Associations. Coburn had been president of the New England Sewage Works Association, chairman of the Sanitary Section of the Boston Society of Civil Engineers and active in the chemists' group of the New England Water Works Association. He had been a Mason, an Odd Fellow and a member of the Ancient & Honorable Artillery Company of Boston.

His wife, Ruth, had died a few months prior to Coburn's own death, but he is survived by a daughter, Mrs. Dorothy Hastings of Wollaston, and by two sons, Stuart R. of Wollaston and William M. Coburn of Littleton, N. H.

Stuart Coburn had many friends throughout the country in the various professional societies to which he belonged, particularly in the groups of chemists and bacteriologists devoted to furthering improvements in treatment of water, sewage and industrial wastes. His keen mind and enthusiasm for discussion of technical subjects are well remembered by those with whom he was long associated.

Although death came as a relief from months of suffering of body and mind, it ended a life of usefulness and accomplishment, and one which under a happier fate could have long continued active in the profession to which it was dedicated.

E. SHERMAN CHASE

MEMOIR OF LAZARUS WHITE

Lazarus White, engineer and contractor, was born at Rochester, N. Y., February 26, 1874, and died July 31, 1953, at his home in Larchmont, N. Y.

He graduated from Columbia University with a C.E. degree in 1897. For a year, he was with the U. S. Engineer Corps and two years with the Pennsylvania Railroad. From 1900 to 1906 he was Section Engineer for the Rapid Transit Commission of New York City. From 1906 to 1914 he was Division Engineer for the Board of Water Supply of New York City, in charge of 15 miles of the Catskill Aqueduct.

In 1915, Mr. White became Managing Engineer for Smith, Houser and McIsaacs, in charge of the construction of a section of the New York City Subway. Here he supervised the support of buildings on foundations 20 feet or more above the bottom of the subway. Leaving this firm in 1919, he organized the engineering and contracting firm of Spencer, White & Prentis, which specialized in foundation work—especially underpinnings—and was its president.

In association with E. A. Prentis, Mr. White invented the underpinning method named the Pretest System. He retired from the presidency in 1949, but remained a member of the firm until his death.

In 1941 he received the Egleston Medal of the Columbia University Alumni Association for "Distinguished Engineering Achievement".

Becoming a member of the American Society of Civil Engineers in 1921, he served as a Director and as Chairman of the Committee on Soils and Foundations. His membership in the New England Water Works Association dated from September, 1913. He was also a member of the American Academy of Science, American Society for Testing Materials, The Society for Ethical Culture and the Engineers Club of New York.

Mr. White was co-author wth Mr. Prentis of "Modern Underpinning", 1917, "Underpinning", 1929, and "Cofferdams", 1940.

He is survived by his widow, two sons and a daughter.

Mr. White was co-author with Mr. Prentis of "Modern Underprinciples in construction work and served as consultant on a number of important projects. He also lectured on foundations at Harvard University.

WILLIAM W. BRUSH

PROCEEDINGS

JANUARY 1954 MEETING

HOTEL STATLER, BOSTON, MASS.

THURSDAY, JANUARY 21, 1954

President William B. Duffy in the Chair.

Secretary Knox announced the election of the following to membership in the Association:

Charles Nagy, Assistant Engineer, Connecticut Light & Power Co., Stafford Springs, Conn.; Edward L. Tyler, Sanitary Engineer, Metropolitan District Commission, Water Division, Fayville Dam, Framingham, Mass.; John H. Duffy, Superintendent, Water Department, Woburn, Mass.; Herbert P. Barton, Foreman, Meter Department, Water Division, Public Works Department, Boston, Mass.; Frederick L. Aldrich, Assistant Superintendent, Water Works, East Providence, R. I.

Associate Members. R. J. Berke & Co., Inc., 34 Manemet Road, Newton Center, Mass.; Mills Engineering Co., Inc., 10 High St., Boston, Mass.

A paper on "Corrosion Control by Metaphosphate Glasses", prepared by Prof. Rolf Eliassen and Prof. James C. Lamb, III, of Massachusetts Institute of Technology, was read by Prof. Lamb.

A paper on "Restrictions on Water Use in Warm Weather" was read by Willard S. Pratt, Director of Public Works, Newton, Mass.

A paper on "How to Secure Maximum Benefits with Activated Carbon" was read by Albert Y. Hyndshaw, Research Chemist, Industrial Chemical Sales Division, West Virginia Pulp and Paper Co., New York, N. Y.

FEBRUARY 1954 MEETING

HOTEL STATLER, BOSTON, MASS.

THURSDAY, FEBRUARY 18, 1954

President William B. Duffy in the Chair.

Secretary Knox announced the election of the following to membership in the Association:

John C. Clifford, Senior Sanitary Engineer, Rhode Island Department of Health, Providence, R. I.; Albert E. Feldman, Assistant Biologist, Water and Sewage Laboratories, Massachusetts Department of Public Health, Boston, Mass.; Harry Larson, Consulting Engineer, Denver, Colo.; Herbert D. Nickerson, Junior Sanitary Engineer, Massachusetts Department of Public Health, Boston, Mass.

A paper on "The West Boylston Water District Water System" was read by Louis C. Campbell, Superintendent, West Boylston Water District, West Boylston, Mass.

A paper on "Increasing the Water Supply for Worcester, Massachusetts" was read by Charles E. Cannon, Project Engineer, Metcalf & Eddy, Boston, Mass.

A paper on "Tastes and Odors in Connecticut Water Supplies" was read by Richard S. Woodhull, Connecticut Department of Health, Hartford, Conn.

March 1954 Meeting

Hotel Statler, Boston, Mass.

Thursday, March 18, 1954

President William B. Duffy in the Chair.

Secretary Knox announced the election of the following to membership:

Clarence R. Day, Assistant Superintendent and Supervisor of Training School, Shirley, Mass.; Henry M. Paynter, Professor of Hydraulic Engineering, Massachusetts Institute of Technology, Cambridge, Mass.

Associate Members. Fischer and Porter Co., Hatboro, Pa.; Globe Rubber Works, Inc., Boston, Mass.

President Duffy announced the death of Prof. Harold K. Barrows, of Massachusetts Institute of Technology.

Upon motion duly made and seconded, the following were elected as a Nominating Committee: Arthur L. Shaw, Donald C. Calderwood, Arthur C. King, Sidney S. Anthony and Thomas R. Camp.

A paper on "Water Supply and Public Health" was read by Dr. Samuel B. Kirkwood, Massachusetts Commissioner of Public Health, Boston, Mass.

A paper on "The Barrington Station of Bristol County Water Company," prepared by C. E. Trowbridge and D. F. Bixler, American Water Works Service Co., Philadelphia, Pa., was read by Mr. Bixler.

A paper on "Recent Improvements to the Shrewsbury Water System," prepared by Lew Brigham, Superintendent, Water Department, Shrewsbury, Mass., was read by Chester A. Culver, Water Commissioner. Colored motion pictures, showing the conditions in Shrewsbury after the 1953 tornado, were shown and explained by Ken Burns, Chief of Police.

AWWA C400-53T

American Water Works Association

Tentative
Standard Specifications
for
Asbestos-Cement Water Pipe

These "Specifications for Asbestos-Cement Water Pipe" are based upon the best known experience and are intended for use under normal conditions. They are not designed for unqualified use under all conditions and the advisability of their use for any installation must be subjected to review by the engineer responsible.

Approved as "Tentative" May 15, 1953

Sixth Printing, May 1954

AMERICAN WATER WORKS ASSOCIATION
Incorporated
521 Fifth Avenue, New York 17, N.Y.

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as a part of the July 1953 Journal AWWA

Tentative Standard Specifications for

Asbestos-Cement Water Pipe

Foreword

These specifications cover asbestos-cement pipe, laid underground in public highways or private rights of way, to be used to distribute municipal water supplies. They cover the minimum requirements for the conditions under which the material is presumed to be used.

The use of the terms "Class 100," "Class 150," and "Class 200" applied to asbestos-cement pipe produced under this document is not intended to imply that the material, when initially installed, is the engineering equivalent of cast-iron pipe bearing the same designation.

With these factors of limitation understood, the Committee on Water Works Practice accepts these specifications as prepared by its Working Committee (S. M. Clarke, chairman) and transmits them for publication to the Board of Directors of AWWA and the Executive Committee of NEWWA.

As the document must remain tentative for at least one year, the Working Committee is continued.

L. R. Howson, *Chairman*
AWWA Committee on Water Works Practice

Sec. 1—Scope

These specifications cover asbestos-cement pipe installed underground, intended for the conveyance of water, in sizes up to and including 36 in. in diameter, and for maximum working pressures of 100, 150, and 200 psi.

Note: In general, it is recommended that, for underground water distribution systems, Class 150 pipe be the lightest used. Local installation and operating conditions may establish other criteria than normal operating pressure for the pipe wall strength. Trench loads, surface loads, excessive water hammer, and corrosive soils are other factors that should be considered.

Sec. 2—Supplementary Details to Be Provided by Purchaser

When pipe is purchased under these specifications, it will be necessary for the purchaser to make supplementary specific statements, for each size and class of pipe, regarding:

(a) Nominal inside diameter.
(b) Class of pipe.
(c) Total laying length of pipe.
(d) Total number, lengths, and extent of machining of short sections for making connections to valves, fittings, or structures, and for making closures.
(e) Specifications for acceptable couplings, if couplings other than the standard ones of the pipe manufacturer are required.
(f) Whether factory inspection by the purchaser will be required.
(g) Whether three-edge bearing tests will be required.

Sec. 3—Definitions

Under these specifications, the following definitions will apply:

3.1. *Purchaser:* The person, firm, corporation, or governmental subdivision entering into a contract or agreement to purchase pipe according to these specifications.

3.2. *Manufacturer:* The person, firm, or corporation that manufactures the pipe.

3.3. *Inspector:* The representative of the purchaser, properly authorized and limited by the particular duties and responsibilities entrusted to him.

3.4. *Inspection:* Inspection of the pipe and of the tests, at the manufacturer's plant, by the inspector.

3.5. *Working pressure:* The maximum hydrostatic pressure to which the pipe will be subjected in normal operation after installation, exclusive of allowance for water hammer.

Sec. 4—Inspection

4.1. This section shall apply only when factory inspection is specified by the purchaser.

4.2. The manufacturer shall notify the purchaser in advance of the inspection and testing of the pipe, in order that the purchaser may be represented at such tests and inspections.

4.3. The inspector shall have free access to those parts of the manufacturer's plant which are concerned with the inspection and testing of the pipe in accordance with these specifications. The manufacturer shall afford the inspector, without charge, all reasonable facilities for determining whether the pipe meets the requirements of these specifications.

4.4. Should the purchaser have no inspector at the manufacturer's plant, the manufacturer shall, if required by the purchaser, furnish a sworn statement that the requirements of these specifications have been met.

4.5. The inspection shall not relieve the manufacturer of his responsibility to furnish and deliver pipe and couplings conforming in all respects to the requirements of these specifications.

Sec. 5—General Requirements

5.1. *Materials.* Pipe shall be composed of an intimate mixture of Portland cement or Portland puzzolana cement and asbestos fiber, with or without the addition of curing agents; shall be free from organic substances; and shall be formed under pressure and thoroughly cured.

5.2. *Classes of pipe.* Pipe shall be made in three classes, for maximum working pressures of 100, 150, and 200 psi, designated as Class 100, Class 150, and Class 200, respectively.

5.3. *Pipe diameters.* Pipe shall be made in nominal inside diameters of 4, 6, 8, 10, 12, 14, 16, 18, 20, 24, 30, and 36 in. For the 4-, 6-, 8-, 10-, and 12-in. sizes, the actual diameters may be less than the nominal by not more than 5.0 per cent, when measured approximately 3 in. from the ends of the pipe.

5.4. *Pipe lengths.* The standard length of pipe shall be not less than 13 ft. At least 90 per cent of the total footage of pipe of any one class and size, excluding short lengths, as specified below, shall be furnished in standard lengths, with a tolerance of ±1 in. The remaining 10 per cent may be in random lengths of not less than 7 ft. All random lengths shall be cut only from standard lengths that have met the requirements of Sec. 8.3.

Short lengths for making connections to valves, fittings, and structures, and for making closures, shall be furnished in such numbers and lengths as specified by the purchaser.

Short lengths shall be not more than 3 ft 3 in. long for the 4-in. and 6-in. sizes, and not more than 6 ft 6 in. long for the 8-in. and larger sizes.

TABLE 1

Flexural Strength

Nom. Diam. *in.*	Total Applied Load—*lb*		
	Class 100	Class 150	Class 200
4	900	1,100	1,400
6	2,100	2,800	3,700
8	4,000	5,700	7,600

The coupling areas of all standard and random lengths shall be machined or otherwise rendered suitable for making a tight joint with the couplings. The short lengths shall be properly machined for their purpose, whether at ends only or over their entire length, as specified by the purchaser.

5.5. *Couplings.* One coupling of the size and class specified shall be furnished with each standard, random, and short length of pipe. Unless otherwise specified by the purchaser, each coupling shall comprise an asbestos-cement sleeve, of the same composition as the pipe, and two rubber rings.

Sec. 6—Detail Requirements

6.1. *Wall thickness.* The wall thickness at the machined portions of any pipe shall not deviate from the manufacturer's standard by more than +0.16 in. or −0.08 in. for pipe having a design thickness of 1 in. or less, nor by more than +0.20 in. or −0.10 in. for pipe having a design thickness greater than 1 in.

6.2. *Straightness.* No pipe shall have a variance from straightness, measured as an outside middle ordinate, of more than 0.625 in. for 13-ft lengths, or of more than a proportionate amount for shorter lengths.

6.3. *Imperfections.* Each pipe shall be free from bulges, dents, and tears on the inside surface that result in a variation in diameter of more than 0.187 in. from the diameter of adjacent, unaffected portions of the surface.

The coupling areas of the barrel of each pipe shall be free from dents and gouges that will affect the tightness of the joint.

The exterior surface or edge of machined ends shall be free from flaking that extends back more than $\frac{1}{2}$ in. from the end, that has a depth of more than $\frac{1}{8}$ in., or that extends around the perimeter more than $\frac{1}{2}$ in. at one location.

Sec. 7—Strength Requirements

7.1. *Bursting strength.* Each pipe and each coupling shall be designed to have sufficient strength to withstand an internal hydrostatic pressure of four times the rated working pressure for the class of pipe.

7.2. *Flexural strength.* Each standard length of pipe of 4-, 6-, and 8-in. nominal diameter shall, when tested in flexure as specified in Sec. 8.3, support the load indicated in Table 1.

7.3. *Crushing strength.* Each pipe shall have sufficient strength, if submitted to the three-edge bearing test as specified in Sec. 8.4, to support not less than the load indicated in Table 2.

TABLE 2

Crushing Strength

Nom. Diam. *in.*	Applied Load per Foot of Length—*lb*		
	Class 100	Class 150	Class 200
4	4,100	5,400	8,700
6	3,900	5,400	9,000
8	3,700	5,500	9,300
10	3,700	7,000	11,000
12	4,000	7,600	11,800
14	4,400	8,600	13,500
16	4,800	9,200	15,400
18	5,200	10,100	17,400
20	5,600	10,900	19,400
24	6,300	12,700	22,600
30	7,500	15,900	28,400
36	8,800	19,600	33,800

Sec. 8—Sampling and Testing Methods

8.1. *Test specimens.* All pipe and couplings tested under this specification shall be in a normal air-dried condition.

8.2. *Hydrostatic tests.* Each standard, random, and short length of pipe and each coupling sleeve shall be tested under an internal hydrostatic pressure as follows:

Class	Pressure *psi*
100	350
150	525
200	700

The water pressure shall be applied gradually and maintained at the specified test level for at least 5 sec. Any pipe or coupling sleeve showing any leakage, sweating, or other defect shall be rejected.

From each 300 lengths of pipe, or fraction thereof, of each size and class, which have passed the routine hydrostatic and flexural tests, one length shall be selected by the inspector. Each selected length shall be hydrostatically tested to a pressure of four times the rated working pressure for the class of pipe, such pressure to be maintained for at least 5 sec. The pipe shall not fail under this pressure. Each pipe so tested shall be retested in the manner specified in the paragraph immediately preceding.

8.3. *Flexure tests.* Each standard length of pipe of 4-, 6-, and 8-in. nominal diameter shall be tested in flexure. The supports shall be 12 ft apart. The total load shall be divided equally and applied at the third points of the clear span. Each pipe so tested shall support, without evidence of cracks or other defects, the applicable total load shown in Table 1. The load shall be applied at a uniform rate.

8.4. *Three-edge bearing tests.* When specified by the purchaser, from each 300 lengths of pipe, or fraction thereof, of each size and class, one length shall be selected by the inspector for a three-edge bearing test. From each selected length, there shall be cut one unmachined section of pipe 1 ft long. This section shall be tested by the three-edge bearing method and shall not fail until the total applied load exceeds the applicable value shown in Table 2. After 75 per cent of the specified load is reached, the load shall be applied at a uniform rate of approximately 2,000 lb per minute.

For this test, the two lower bearings shall consist of two straight wooden strips with vertical sides, each strip having its interior top edge rounded to a radius of approximately $\frac{1}{2}$ in. The strips shall be securely fastened to a rigid block, with the interior vertical faces parallel and a distance apart of not less than $\frac{1}{2}$ in. nor more than 1 in. per foot of diameter of the pipe. The upper bearing shall be a rigid wooden block, at least 6 in. by 6 in. in cross section, straight and true from end to end. The upper and lower bearings shall extend the full length of the test section.

8.5. *Machines for testing.* The machine used for the hydrostatic tests shall have gaskets that seal the ends of the pipe without materially counteracting the hydrostatic test pressure.

The machines used for the flexure and three-edge bearing tests shall be substantial and rigid throughout, so that the distribution of the load will not be appreciably affected by the deformation or yielding of any part of the machine.

8.6. *Retests and rejection.* The failure of any specimen tested for crushing strength to support 75 per cent of the crushing load required in Sec. 7.3 shall

be cause for rejection of the entire lot of that size and class manufactured during the same shift as the test specimen.

If any specimen tested for crushing strength supports more than 75 per cent but less than 100 per cent of the crushing load required in Sec. 7.3, one specimen shall be cut from each of five additional pipe sections of the same size and class manufactured during the same shift and shall be subjected to the crushing test. The failure of two of these additional specimens to meet the crushing strength requirements of Sec. 7.3 shall be cause for rejection of the entire lot of that size and class manufactured during the same shift as the test specimens.

If any pipe subjected to the higher hydrostatic test fails to withstand the specified pressure, five additional lengths of the same size and class shall be selected from the pipe manufactured during the same shift and shall be subjected to the higher hydrostatic test. The failure of two of these additional lengths to withstand the specified pressure shall be cause for rejection of the entire lot of that size and class manufactured during the same shift as the test lengths.

Sec. 9—Marking

9.1. Each standard and random length of pipe shall be clearly marked on the outside surface with the trade name, nominal inside diameter, class, hydrostatic test pressure, and date and shift of manufacture.

9.2. Each short length of pipe shall be clearly marked on the outside surface with the nominal inside diameter, the class, and the letter *T* to indicate that it has been hydrostatically tested.

9.3. All component parts of each coupling shall be clearly marked with the make and the size and class of pipe for which they are intended. Each coupling sleeve shall also be marked with the letter *T* to indicate that it has been hydrostatically tested.

9.4. When factory inspection is made by the purchaser, each pipe and each coupling sleeve shall receive an additional special marking of no more than three letters, as required by the purchaser.

Sec. 10—Preparation for Shipment

All pipe and couplings shall, unless otherwise specified, be prepared for standard commercial shipment, so as to insure acceptance by common or other carriers.

AWWA C601-53T
(*Formerly* 7D.2)

American Water Works Association

A Tentative Procedure

for

DISINFECTING WATER MAINS

With Comments by Committee Chairman

This "Procedure for Disinfecting Water Mains" is based upon competent experience and is intended for use under normal conditions. The procedure is not intended for unqualified use under all conditions.

Approved as Revised by AWWA Mar. 6, 1953

First Edition: 7D.2–1948 (C601–48); Revised Edition: C601–53T
Fourth Printing, May 1954

AMERICAN WATER WORKS ASSOCIATION
Incorporated
521 Fifth Avenue, New York 17, N.Y.

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C. F. Wertz

"A Procedure for Disinfecting Water Mains" was originally adopted by AWWA on Sept. 30, 1947, and published in the February 1948 Journal. A minor revision was made on Sept. 15, 1948. The present text represents a complete revision of the entire procedure and was approved by the AWWA Board of Directors on Mar. 6, 1953.

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as a part of the August 1953 Journal AWWA

Foreword

It is the intent of this document to present a practical and workable main disinfection procedure suitable for application under any reasonably normal conditions. Published records show that no procedure for the disinfection of newly laid water mains has been universally or continuously satisfactory. In the water in newly laid mains, types of bacteria that are indicative of pollution may be found. Since laboratory tests cannot determine their origin, it is necessary to take all possible precautions to keep them out, or to destroy them where tests show them to be present.

The revised procedure in no way reduces the water quality requirements before the main can be placed in service. In all instances, the quality must be proved by satisfactory samples.

Users of this procedure are cautioned that there is a need for thorough displacement of strong disinfectant solutions from the mains and for keeping such solutions from entering service lines. Illness or discomfort and damage to household plumbing or fixtures may result from the use of water containing strong disinfectants.

Some metals may be damaged by these solutions. It is solely the responsibility of the person acquainted with the local conditions to recognize and guard against these hazards.

No procedure for disinfection of jute is included, because jute is not listed as acceptable in the Specifications for Installation of Cast-Iron Water Mains—AWWA C600.

It may be found advantageous to combine the disinfection of a newly laid pipeline with the hydrostatic tests required under Sec. 13 of AWWA C600. The decision concerning the combining of such operations is, of course, the prerogative of the engineer in charge of the installation of the new main. Combining disinfection and the hydrostatic test may, however, sacrifice the advantage of an initial flushing out of the lighter solids and particulate matter having a chlorine demand. Repetition of the disinfection procedure may be required, resulting in an overall delay.

The development of this procedural document by the Association implies the responsibility of the water works executive for all steps in the disinfection procedure, including the sampling and laboratory examination of water from the main after disinfection has been completed. It is assumed that the health authorities, at either the local or state (provincial) level, maintain a cooperative interest in the operations. If the water works executive desires to relinquish to the health authorities his responsibility for sampling and laboratory examination of water from the disinfected mains, there is no reason why he should not do so.

A Tentative Procedure for

Disinfecting Water Mains

Sec. 1—Pipe Kept Clean

The interior of all pipe, fittings, and other accessories shall be kept as free as possible from dirt and foreign matter at all times.

Sec. 2—Pipe to Be Cleaned and Treated

When the pipe is laid, it shall be as free as possible of all foreign matter. If, in the opinion of the engineer or job superintendent, the pipe contains dirt that will not be removed during the flushing operation, the interior of the pipe shall be cleaned and swabbed, as necessary, with a bactericidal solution.

Solutions for swabbing may be made from the compounds listed in Sec. 10.1. No other compound shall be used unless it is approved by the local and state health authorities.

Sec. 3—Laying Pipe

Every precaution shall be used to protect the pipe against the entrance of foreign material before the pipe is placed in the new line. At the close of the day's work, or whenever the workmen are absent from the job, the end of the last laid section of pipe shall be plugged, capped, or otherwise tightly closed to prevent the entry of foreign material of any nature.

If the contractor, or pipe-laying crew, cannot put the pipe into the trench, and in place, without getting earth into it, the engineer may require that, before lowering the pipe into the trench, a heavy, tightly woven canvas bag of suitable size shall be placed over each end and left there until the connection is to be made to the adjacent pipe.

Sec. 4—Preventing Trench Water From Entering Pipe

At times when pipe laying is not in progress, the open ends of pipe shall be closed by a watertight plug or other means approved by the engineer. Joints of pipe in the trench shall be poured before the work is stopped. If water is in the trench, the seal shall remain in place until the trench is pumped dry.

Sec. 5—Packing Material

This procedure implies the use of only such packing material as is included in the list of acceptable materials in the Specifications for Installation of Cast-Iron Water Mains—AWWA C600. Packing material shall be handled in a manner to avoid contamination and shall be dry when placed in the joint. All such material shall be free of oil, tar, or greasy substances.

Sec. 6—Flushing Completed Pipelines

6.1. *Preliminary flushing.* The main shall be flushed, prior to chlorination, as thoroughly as possible with the water pressure and outlets available. Flushing shall be done after the pressure test has been made. It must be

understood that flushing removes only the lighter solids and cannot be relied upon to remove heavy material allowed to get into the main during laying (see Sec. 2). If no hydrant is installed at the end of the main, a tap should be provided large enough to develop a velocity in the main of at least 2.5 fps (see Table 1 for openings required to flush pipelines at a 2.5-fps velocity).

NOTE: It will be found difficult to flush mains over 16 in. in diameter at the 2.5-fps rate. In such instances, the requirements of Sec. 2, 3, and 4 must be rigidly enforced.

6.2. *Valve damage by foreign material.* Unless proper care and thorough inspection are practiced during the laying of water mains, small stones, pieces of concrete, particles of metal, or other foreign material may gain access to mains newly laid or repaired. If it is believed that such foreign material may be in the main, all hydrants on the line shall be thoroughly flushed and carefully inspected after flushing to see that the entire valve-operating mechanism of each hydrant is in good condition.

TABLE 1

Required Openings to Flush Pipelines
(40-psi Pressure)

Pipe Size *in.*	2.5-fps Flushing Velocity				4-fps Flushing Velocity		
	Flow *gpm*	Orifice Size *in.*	Hydrant Openings		Flow *gpm*	Hydrant Openings	
			No.	Size *in.*		No.	Size *in.*
4	100	$1\frac{1}{16}$	1	$2\frac{1}{2}$	160	1	$2\frac{1}{2}$
6	220	$1\frac{3}{8}$	1	$2\frac{1}{2}$	350	1	$2\frac{1}{2}$
8	390	$1\frac{7}{8}$	1	$2\frac{1}{2}$	630	1	$2\frac{1}{2}$
10	610	$2\frac{5}{16}$	1	$2\frac{1}{2}$	980	1	$2\frac{1}{2}$
12	880	$2\frac{13}{16}$	1	$2\frac{1}{2}$	1,400	2	$2\frac{1}{2}$
14	1,200	$3\frac{1}{4}$	2	$2\frac{1}{2}$	1,920	2	$2\frac{1}{2}$
16	1,565	$3\frac{3}{8}$	2	$2\frac{1}{2}$	2,510	3	$2\frac{1}{2}$
						or: 1	$4\frac{1}{2}$
18	1,980	$4\frac{3}{16}$	2	$2\frac{1}{2}$	3,180	3	$2\frac{1}{2}$
						or: 1	$4\frac{1}{2}$
20	2,450	$4\frac{5}{8}$	3	$2\frac{1}{2}$	3,930	4	$2\frac{1}{2}$
			or: 1	$4\frac{1}{2}$		*or:* 1	$4\frac{1}{2}$
						1	$2\frac{1}{2}$
24	3,510	$5\frac{9}{16}$	4	$2\frac{1}{2}$	5,650	6	$2\frac{1}{2}$
			or: 1	$4\frac{1}{2}$		*or:* 1	$4\frac{1}{2}$
			1	$2\frac{1}{2}$		2	$2\frac{1}{2}$
						or: 2	$4\frac{1}{2}$
30	5,500	7	5	$2\frac{1}{2}$	8,830	3	$4\frac{1}{2}$
			or: 1	$4\frac{1}{2}$		*or:* 2	$4\frac{1}{2}$
			2	$2\frac{1}{2}$		2	$2\frac{1}{2}$
			or: 2	$4\frac{1}{2}$			

Sec. 7—Requirement of Chlorination

Before being placed in service, all new mains and repaired portions of, or extensions to, existing mains shall be chlorinated so that a chlorine re-

sidual of not less than 10 ppm remains in the water after 24 hours' standing in the pipe. Exemption from the requirement of chlorination may be granted by the public health authority having jurisdiction, if distribution system chlorine residuals persist throughout the new main and are of sufficient magnitude so that samples of the water delivered by the new main meet the approval of the public health authority.

Sec. 8—Form of Applied Chlorine

Any of the following methods of procedure (arranged in order of preference) shall be followed, subject to the approval of the engineer:

Liquid chlorine gas–water mixture	Sec. 9
Direct chlorine feed	Sec. 9
Calcium or sodium hypochlorite and water mixture	Sec. 10
Chlorinated lime and water mixture	Sec. 10

Sec. 9—Liquid Chlorine

A chlorine gas–water mixture shall be applied by means of a solution-feed chlorinating device, or, if approved by the engineer, the dry gas may be fed directly through proper devices for regulating the rate of flow and providing effective diffusion of the gas into the water within the pipe being treated. Chlorinating devices for feeding solutions of the chlorine gas or the gas itself must provide means for preventing the backflow of water into the chlorine cylinder.

Sec. 10—Chlorine-bearing Compounds in Water

On approval of the engineer, a mixture of water and a chlorine-bearing compound of known chlorine content may be substituted for liquid chlorine.

10.1. *Compounds to be used.* The chlorine-bearing compounds that may be used are: [1] calcium hypochlorite,* [2] chlorinated lime,† and [3] sodium hypochlorite.‡

The use of chlorinated lime (which is low in available chlorine content) increases the sludge problem and may interfere with feeder operation. Exposure to the atmosphere and sunlight results in the loss of chlorine before or after being mixed with water.

10.2. *Preparation of mixture.* High-test calcium hypochlorite or bleaching powder must be prepared as a water mixture for introduction into the water mains. The powder should first be made into a paste and then thinned to approximately a 1 per cent chlorine solution (10,000 ppm). The preparation of a 1 per cent chlorine solution requires the following proportions of powder to water:

Product	Amount of Compound	Quantity of Water gal
High-test calcium hypochlorite (65–70% Cl)	1 lb	7.50
Chlorinated lime (32–35% Cl)	2 lb	7.50
Liquid laundry bleach (5.25% Cl)	1 gal	4.25

Table 2 gives the amount of chlorine required for each 100 ft of pipe of various sizes.

Sec. 11—Point of Application

The preferred point of application of the chlorinating agent is at the begin-

* Comparable to commercial products known as "HTH," "Perchloron," and "Pittchlor."

† Frequently called chloride of lime and known to industry as "bleaching powder."

‡ Known commercially as "liquid laundry bleach."

ning of the pipeline extension or any valved section of it and through a corporation stop inserted by the owner (except in new distribution systems) in the top of the newly laid pipe. The water injector for delivering the chlorine-bearing water into the pipe should be supplied from a tap on the pressure side of the gate valve controlling the flow into the pipeline extension. In a new system, application of chlorine may be made advantageously at the pumping station, the elevated tank, the standpipe, or the reservoir.

TABLE 2

Chlorine Requirements for 100-ft Lengths of Various Sizes of Pipe

Pipe Size *in.*	Volume of 100-ft Length *gal*	Amount Required to Give 25 ppm Cl	
		100% Chlorine *lb*	1% Chlorine Water Solution *gal*
4	65.3	0.0135	$\frac{1}{2}$
6	146.5	0.0305	$\frac{3}{4}$
8	261.0	0.054	$\frac{2}{3}$
10	408.0	0.085	1
12	588.7	0.120	$1\frac{1}{2}$

When properly cleaned first, these units are thus chlorinated adequately.

Sec. 12—Rate of Application

Water from the existing distribution system or other source of supply shall be controlled so as to flow slowly into the newly laid pipeline during the application of chlorine. The rate of chlorine mixture flow shall be in such proportion to the rate of water entering the pipe that the chlorine dose applied to the water entering the newly laid pipe shall produce at least 10 ppm after 24 hours' standing. This may be expected with an application of 25 ppm, although some conditions may require more. In application, those persons familiar with the normal chlorine demand of the water and the conditions of the job are best qualified to determine the dosage required. When the results obtained are not in line with past experiences, however, they should be interpreted as evidence that swabbing of the pipe before installation or that flushing of the pipe was inadequately performed.

A convenient method of determining the rate of flow of water into the line to be treated is to start with the line full of water and measure the rate of discharge at a hydrant with a pitot gage. Greater flexibility is made possible by providing a series of orifices to give accurate gage readings at high and low flows.

Sec. 13—Preventing Reverse Flow

Valves shall be manipulated so that the strong chlorine solution in the line being treated will not flow back into the line supplying the water. Check valves may be used, if desired.

Sec. 14—Retention Period

Treated water shall be retained in the pipe long enough to destroy all nonspore-forming bacteria. This period should be at least 24 hours and should produce no less than 10 ppm at the extreme end of the line at the end of the retention period.

NOTE: If the circumstances are such that a shorter retention period must be used, the chlorine concentration shall be increased accordingly. For instance, for a contact period of 1 hour, a 100-ppm chlorine concentration is required. Under these conditions, special care should be taken to avoid attack on

pipes, valves, hydrants, and other appurtenances.

Sec. 15—Chlorinating Valves and Hydrants

In the process of chlorinating newly laid pipe, all valves or other appurtenances shall be operated while the pipeline is filled with the chlorinating agent.

Sec. 16—Final Flushing and Test

Following chlorination, all treated water shall be thoroughly flushed from the newly laid pipeline at its extremities until the replacement water throughout its length shall, upon test, be proved comparable in quality to the water served the public from the existing water supply system and approved by the public health authority having jurisdiction. This satisfactory quality of water delivered by the new main should continue for a period of at least 2 full days as demonstrated by laboratory examination of samples taken from a tap located and installed in such a way as to prevent outside contamination. Samples should never be taken from an unsterilized hose or from a fire hydrant, because such samples will seldom meet bacteriological standards.

Sec. 17—Repetition of Procedure

Should the initial treatment fail to result in the conditions specified in Sec. 16, the original chlorination procedure shall be repeated until satisfactory results are obtained. *Or:*

A free available chlorine residual of not less than 0.4 ppm shall be maintained throughout the entire length of the new main. This permits the immediate use of water from the new main so long as an 0.4-ppm free available chlorine residual is present. The treatment shall continue until the samples on two successive days shall be comparable in quality to the water served the public from the existing water supply system. *Or:*

A combined available chlorine residual of not less than 1.0 ppm shall be maintained throughout the entire length of the new main. This permits the immediate use of water from the new main so long as 1.0 ppm combined available chlorine is present. The treatment shall continue until samples on two successive days shall be comparable in quality to the water served the public from the existing water supply system.

Sec. 18—Procedure When Repairing or Cutting Into Existing Lines

18.1. *General.* The location of a break cannot be forecast. Distribution valve spacing and the position of cutins and breaks relative to hydrants present a complexity of combinations. Only a thoroughly trained, ingenious, and experienced person should be placed in charge of disinfection following the opening of a water main, whether the opening was accidental or by design.

18.1.1. *Trench pollution.* Whether an old line is opened by accident or design, the excavation is always wet and frequently badly contaminated. The likelihood of serious contamination should be taken into account in planning disinfection.

NOTE: The procedure outlined in this section applies in particular to breaks that involve partial or total dewatering of mains. In the event of minor leaks or breaks that are repaired with clamping devices while the mains remain full of water under pressure, the leak should remain exposed while the clamping device is applied. The gasket, which comes in contact with the break, shall be properly disinfected

with a strong hypochlorite solution before it is clamped to the pipe.

18.1.2. *Need for haste.* There is always need for haste in putting an old line back into service. The importance of the service rendered by the line is a significant factor in determining the methods of disinfection that ought to be used.

18.2. *Pipe and packing material.* Packing material, pipes, fittings, and valves should be handled and treated in the same manner as recommended for new mains under Sec. 1 through Sec. 5. The minimum recommended treatment is swabbing the interior of the pipe or fitting with a strong (20–25 ppm) solution of hypochlorite.

18.3. *Flushing.* Flushing shall always follow the closing of an opening in a main; however, a reasonable contact time with the disinfectant shall precede the flushing operation. If valving and hydrant locations permit, flushing from two directions is helpful. Flushing should be continued until discolored water is eliminated. Flushing through existing hydrants is most convenient, but unusual contamination hazards may require the installation of special flushing taps.

18.4. *Disinfection.* In locations where the direction of flow is known, and where tests show that the water which will flow into the newly installed pipe and fittings contains a substantial free available chlorine residual (approximately 0.5 ppm), it is probable that no contamination will reach a customer. Proximity of customers' taps and the resulting contact period must be taken into consideration.

18.4.1. *Application of chlorine.* Liquid chlorine is the preferred form to be applied, but hypochlorites may be used. The simplest, but least effective, means of disinfection is the direct application of dry hypochlorite, or a heavy slurry of it, on either side of the opening, before the new pipe and fittings are installed.

18.4.2. *Introduction of chlorine.* The solution of chlorine or the suspension of hypochlorites may be introduced through a tap made for the purpose or, more conveniently, through a fire hydrant, when one is available in a suitable location.

18.4.3. *Distribution and flushing.* The chlorine should be added in such a way as to insure that a high concentration (20–25 ppm) reaches every part of the isolated section of the main.

18.4.4. *Contact time and flushing.* The maximum permissible contact period should be used, after which the water bearing strong chlorine should be flushed out before the main is returned to service.

18.4.5. *Sampling.* In order to develop confidence in the efficacy of the procedures employed and to provide a protective historical record, sampling and laboratory examination of the water should be carried on for some days, as determined by the engineer in charge. A single sample taken immediately after chlorination is not informative unless the results are unfavorable.

The following comments, although not a part of the procedure for disinfecting water mains, are reprinted here for the interest of users of the document.

Revised Main Disinfection Procedure

By Marshall P. Crabill

A contribution to the Journal by Marshall P. Crabill, Chairman, Committee E7.D2—Procedure for Disinfecting Water Mains; Supt. of Purif., Indianapolis Water Co., Indianapolis, Ind.

THE Committee on Main Disinfection, in revising the procedural document (under same cover) has endeavored to make it both practical and workable. The first consideration in this or any other procedure or specification is to weigh the need for it. No one will deny the need for the disinfection of newly laid mains, or repaired old ones, where the samples of the water delivered fail to meet public health standards. That is the one and only basis of the need for disinfection. Other conditions, such as tastes or odors, may require treatment, perhaps chlorination, but not for the purpose of disinfection. The committee presents the most practical method known at present to assist in furnishing a water that will meet public health requirements.

It is recognized that sometimes, or perhaps often, a newly laid main may not require more than flushing. It is not unreasonable to acknowledge that, under certain conditions of treatment, some waters materially reduce the possibility of aftergrowth in the distribution system. Neither is it unreasonable to acknowledge that some newly laid pipelines are handled a great deal more carefully and with greater regard for cleanliness than others. Some types of lining and some methods of joining pipe lengths tend to improve the possibilities of recovering satisfactory samples. Extreme care and cleanliness cannot, however, be accepted as prima facie evidence that disinfection is not needed. That evidence can be gathered only on the basis of actual sampling with satisfactory results determined in the laboratory. Exception to the need for disinfection is recognized in Sec. 7 of the procedure.

Many water utilities are not equipped with laboratory facilities to determine whether or not the samples from the mains meet public health standards. Under these conditions, the committee feels that the decision on the need, or the lack of need, for disinfection should be made by the public health authority having jurisdiction in the area of the utility. The decision can be made either on the basis of sample results submitted to the authority by the utility, or on samples collected and tested by the health authority. Sampling should be done with full regard to proper standing time in the new main, and a single sampling should not be considered complete evidence.

The revised procedure leaves the decision on the need for swabbing a new pipe to the person directing the main-laying program. It seems obvious that some pipe will require swabbing with a disinfecting solution and that some will not. One has only to look at the pipe

interior before it is lowered into the trench to determine whether or not flushing is likely to remove the incrusted matter. If not, there is a definite need for swabbing. Plainly, the only person who is qualified to make that decision is the person on the job. The committee feels that, if inspections are made in a conscientious and workmanlike manner, swabbing will be done when and if it is essential.

For any job other than an unusual one, the magnitude of chlorine application and chlorine residual remaining after the recommended contact time has been reduced in the revised procedure. This reduction has the advantage of permitting the use of smaller-capacity equipment, which, for many utilities, may mean that existing equipment will serve satisfactorily. It also permits more rapid bleeding of diluting water into the new main, which tends to reduce the overall time required to apply the disinfectant. The size of the chlorine application was not decreased without trial, and the results on jobs where lower applications were used equaled those with higher applications.

In World War II, the British adopted a much lower residual than the earlier AWWA procedure as being more practical and equally as efficient in meeting the demands of adequate disinfection. No evidence is reported that the lower applications were not satisfactory.

The revised procedure departs from the original in implying the use of only such packing material as is approved in the Specifications for Installation of Cast-Iron Water Mains—AWWA C600. This establishes a uniformity between two documents that are in fact closely associated. No method for treating unapproved packing materials is given. All of the methods studied in the past produce variable results and cannot be defended.

To be effective in removing the lighter solid particles, the velocity of the flushing water must be sufficient to pick up and transport these particles to the discharge point. The committee attaches considerable importance to maintaining a velocity of not less than 2.5 fps through the main being flushed. This should not be assumed to be a scouring velocity and should not be used as an excuse to avoid swabbing heavy incrusted matter. If there is heavy material in the main, swabbing is essential and no advantage will result from avoiding it. Flushing can be depended upon to remove only the lighter solids, even if the recommended velocity is maintained.

The procedural document emphasizes that bacteriological quality cannot be judged from samples collected from hydrants. This warning should not be ignored. In many disinfection jobs, coliform organisms have been found in all samples collected at a hydrant, although totally absent in samples taken 50 ft away through a riser pipe installed on a corporation cock. The riser pipe, if it was not installed at a time when it could be filled with disinfectant solution from the main, was, of course, disinfected separately.

The only disinfecting agent recognized in the procedure is chlorine in one of several forms. This does not preclude the use of another disinfecting agent if it has been tried and is approved by the local health authorities. Several disinfectants other than chlorine have been found entirely satisfactory, particularly in the swabbing operation. The committee could not, however, undertake to test all the other disinfectants.

Indianapolis Experience

The author's utility was recently faced with an unusually difficult disinfection problem. A main approximately 34,000 ft long had to be laid from the pumping station to the site of a large plant manufacturing electrical equipment. A creek crossing was involved, and the main was contaminated with trench water on several occasions during the laying. The plant had its own laboratory for making routine bacteriological determinations on the water, for industrial and drinking use, which was to be delivered from the main to various parts of the plant by a small, plant-owned distribution system.

The situation demanded full utilization of all available knowledge to make the disinfection really successful. Fortunately, mechanical-joint pipe was employed. The main was chlorinated in sections, as completed, and then water with high chlorine residuals was flushed through every hydrant in the line. It was necessary to rechlorinate the creek crossing and adjacent sections before satisfactory samples could be recovered. In no instance was a satisfactory sample collected from a hydrant, although special sampling risers nearby produced good results. The piping at a new automatic booster station in the utility's system also had to be chlorinated. The last section of main was chlorinated within a few days after the laying had been completed, and satisfactory samples were obtained throughout the entire line. Following the utility's example, the manufacturing plant decided to disinfect its own distribution system. The utility did not take part in this phase of the work except to offer advice and check the bacteriological results. The disinfection of the intraplant system was accomplished rapidly and successfully.

It is, of course, impossible to estimate how much illness or discomfort was avoided through disinfection of the mains, but it is gratifying to the utility to know that it is furnishing this industry, and all other consumers, with a water meeting public health standards. Thus, the utility is discharging its obligation to employ every available means in providing a safe and palatable water.

Conclusion

There is still much to be learned about main disinfection. The committee does not believe that the revised procedure will remain in its present form indefinitely. If it does, this fact will indicate that it is being disregarded, as it can be improved only through constructive criticism based on the experience gained by water works men using the procedure. There is substantial evidence, however, that the revised procedure is workable, and, at present, no evidence exists that it will not produce satisfactory results.

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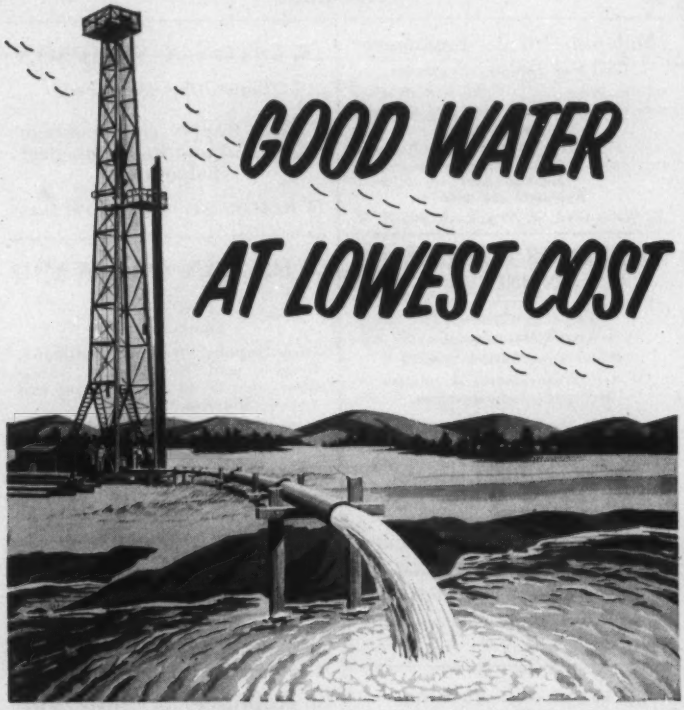

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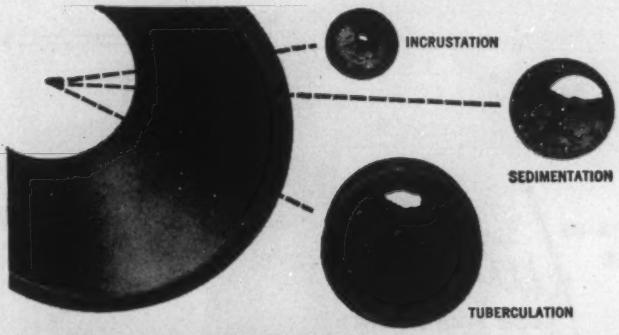

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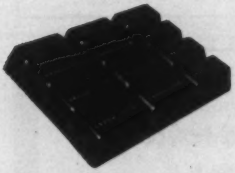

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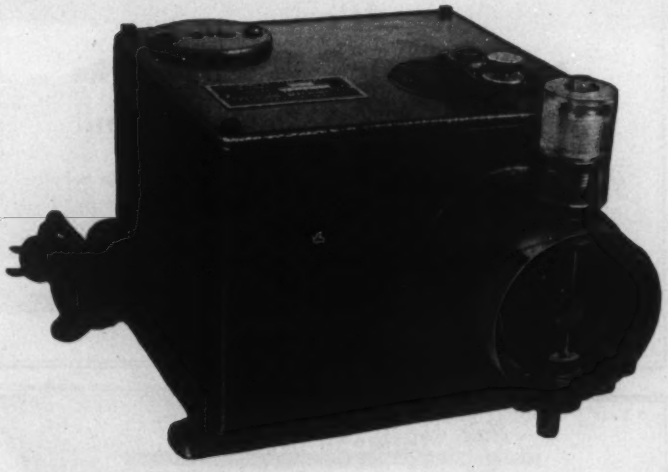

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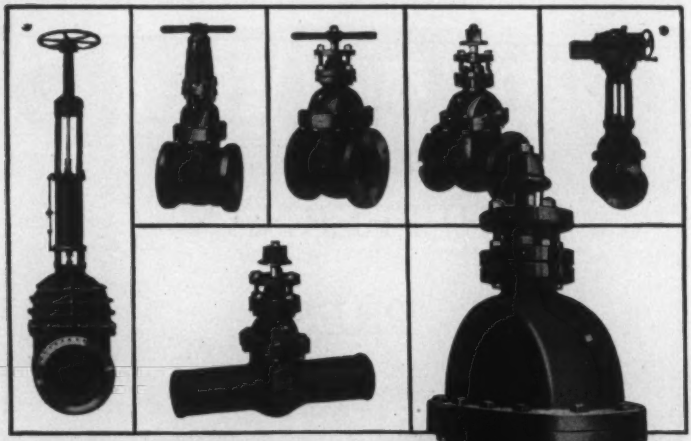

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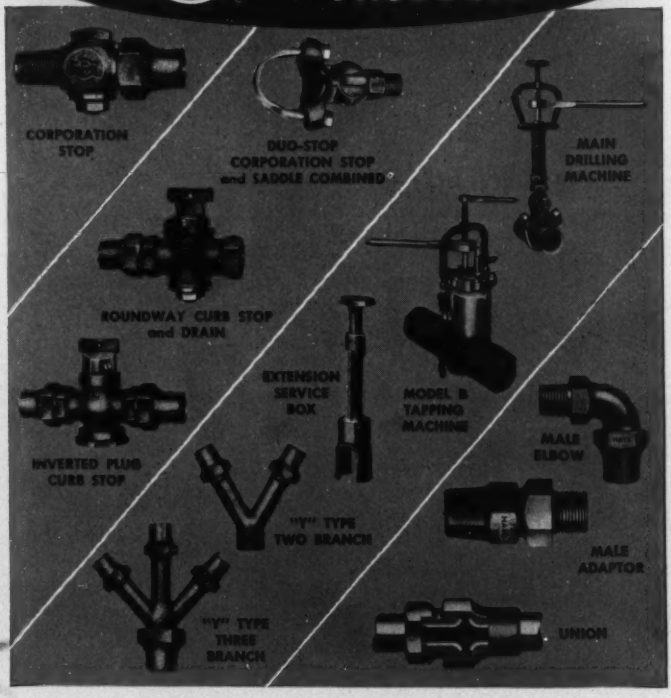

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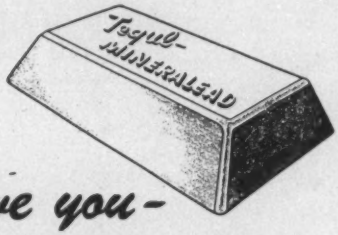

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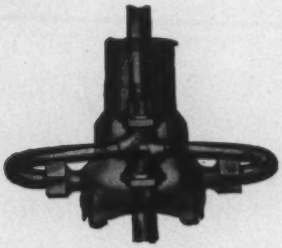

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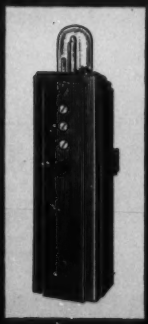

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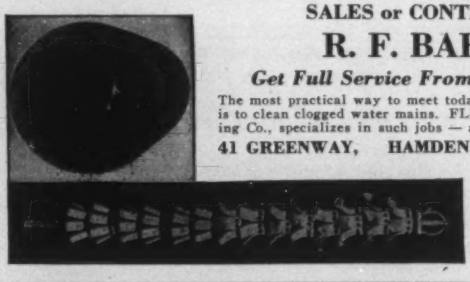

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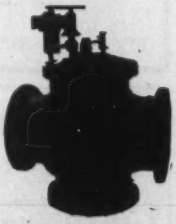

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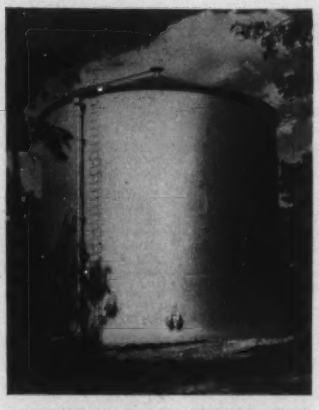

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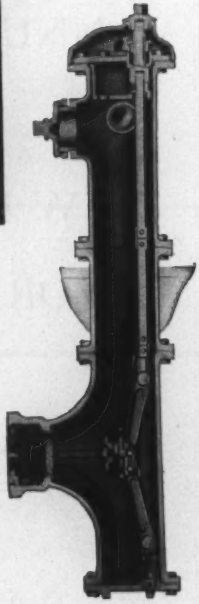

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